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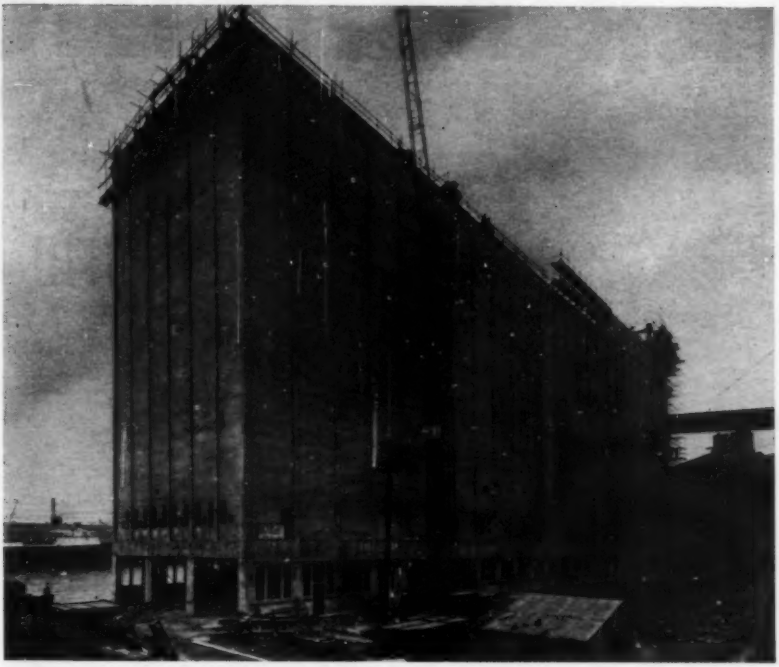

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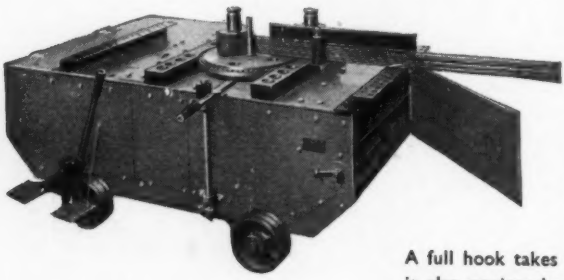

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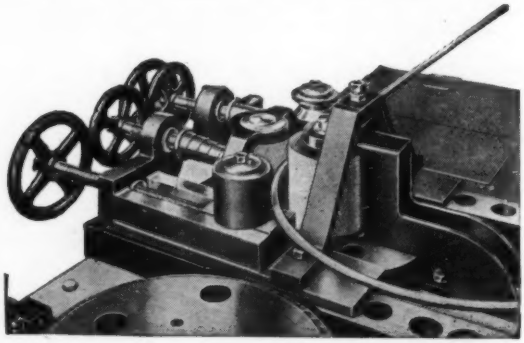

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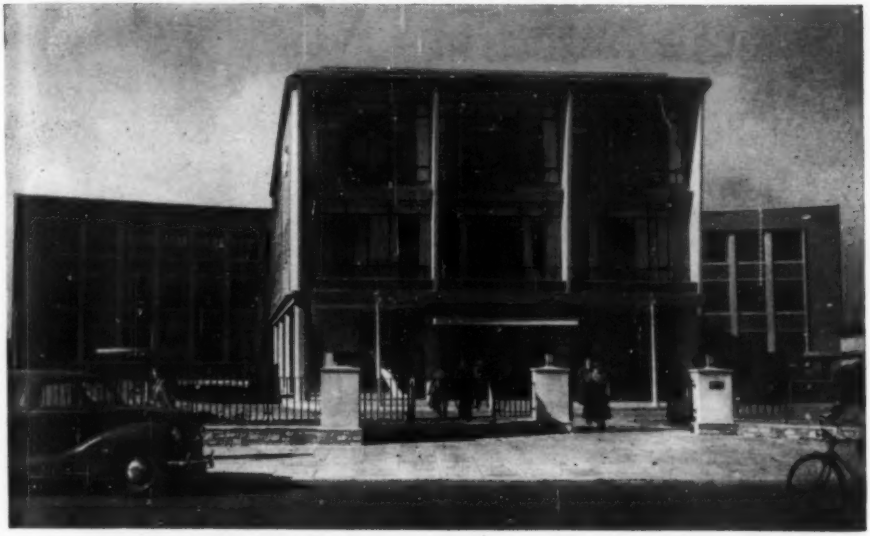

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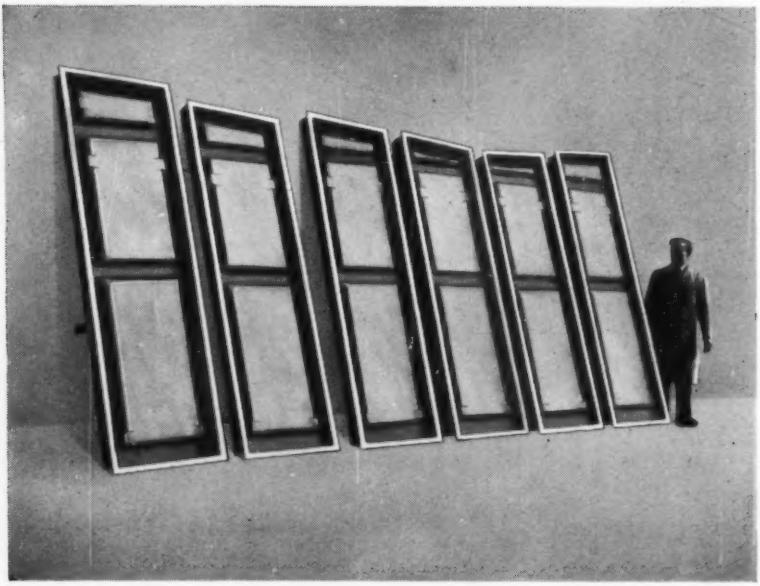

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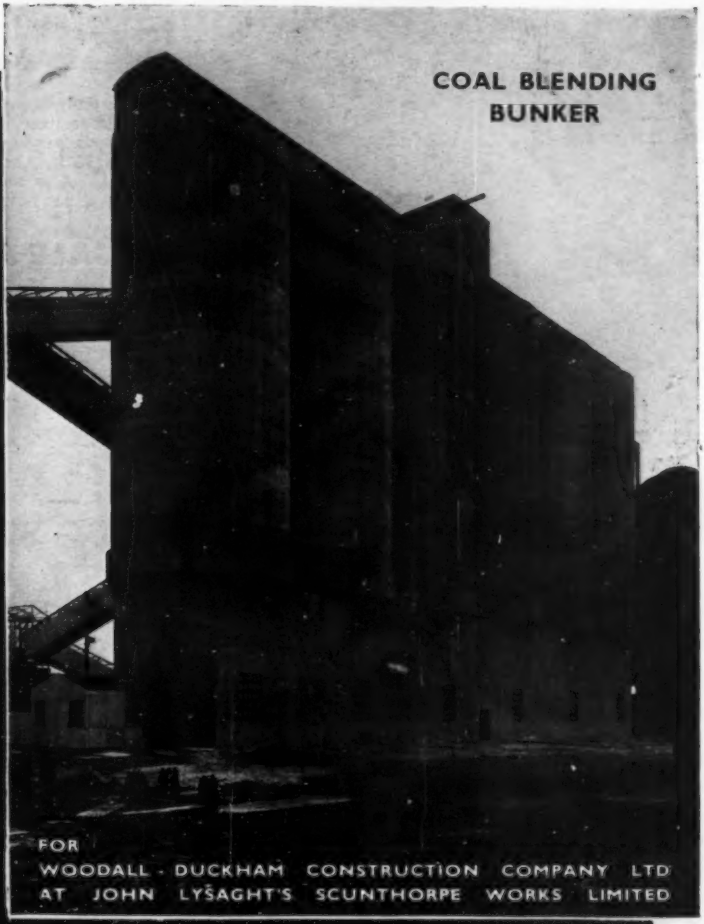

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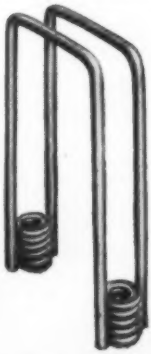

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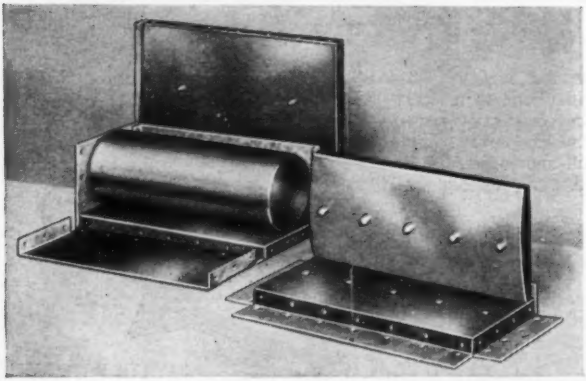
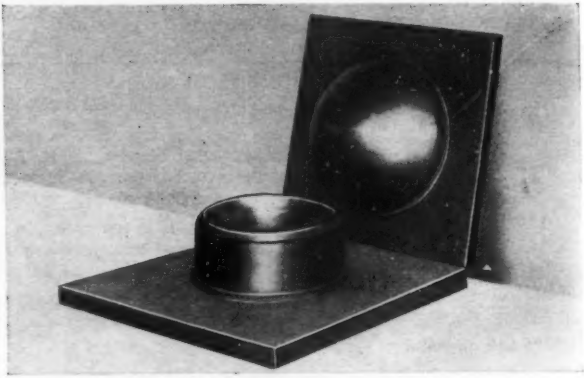
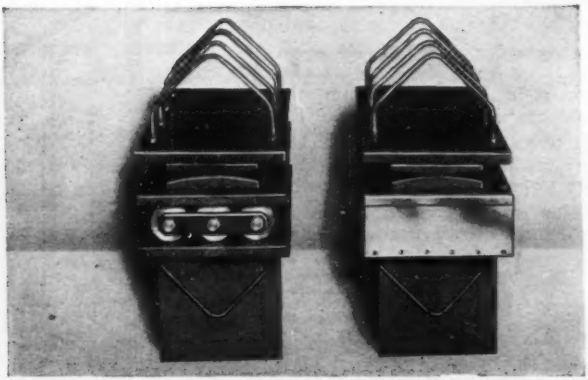


Illustrated are a few examples of various Bridge Bearings recently constructed by Westwoods.





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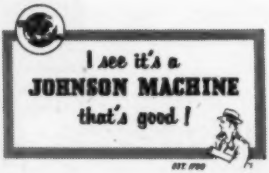

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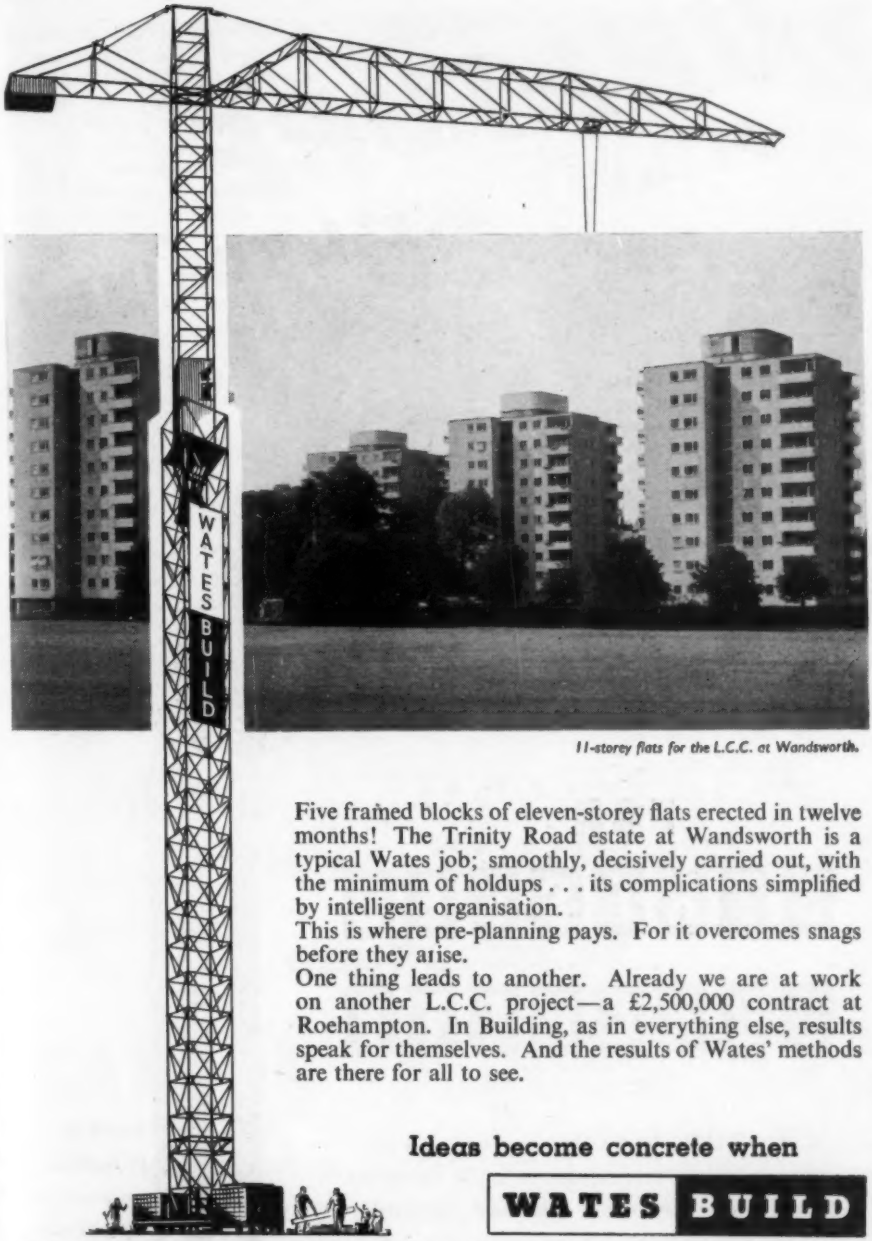

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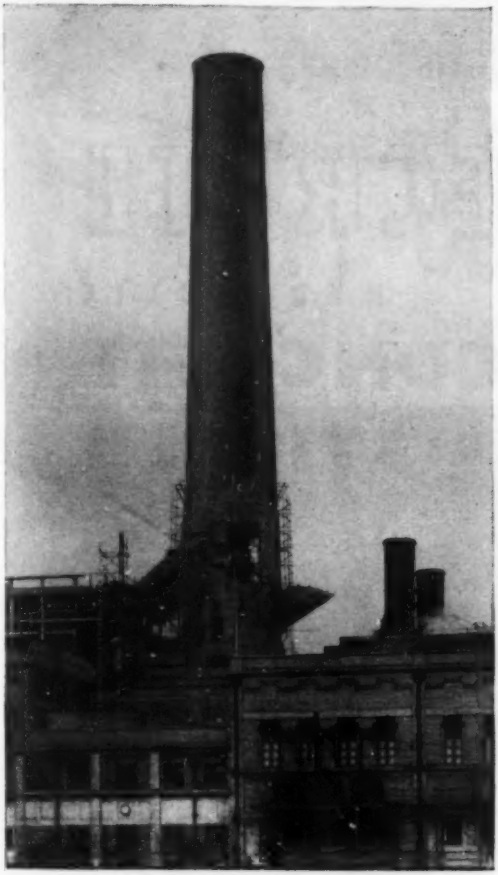

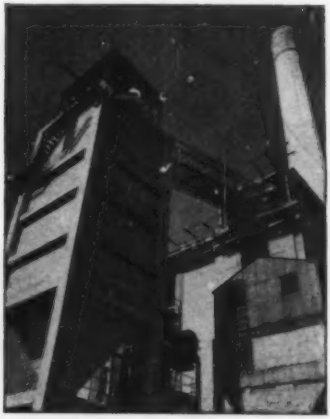

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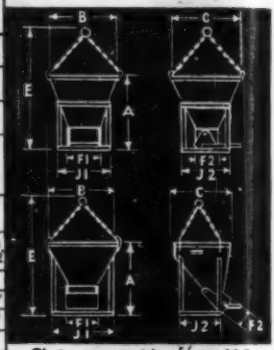

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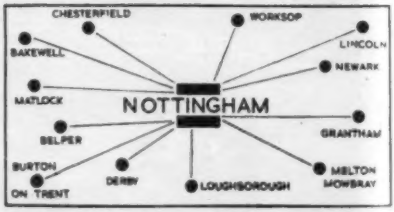

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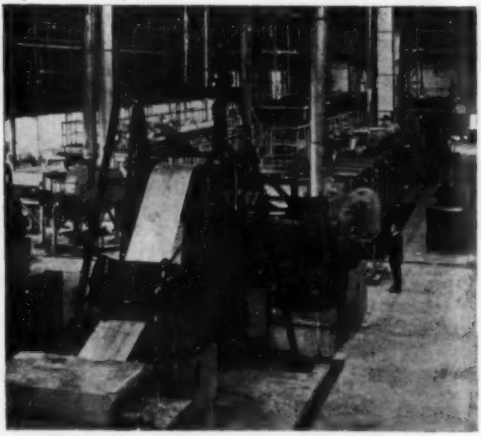

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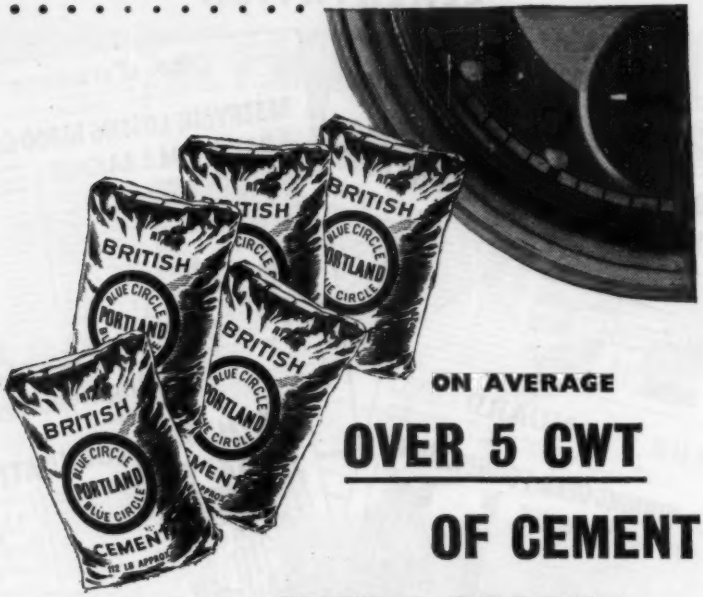

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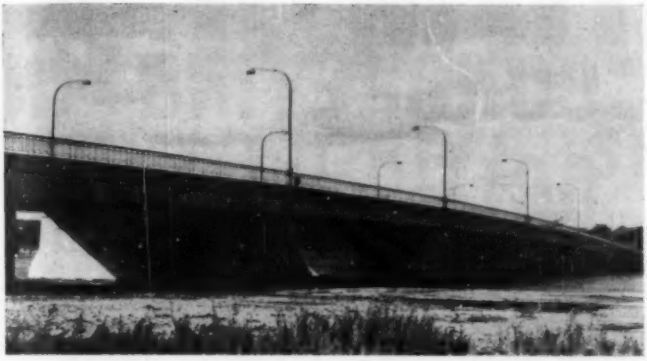

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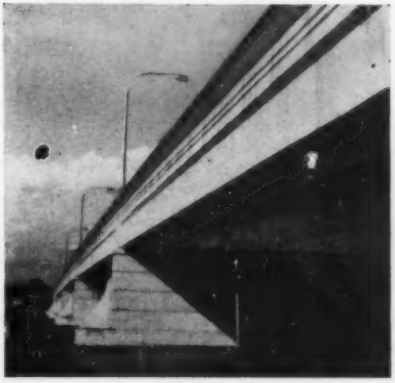

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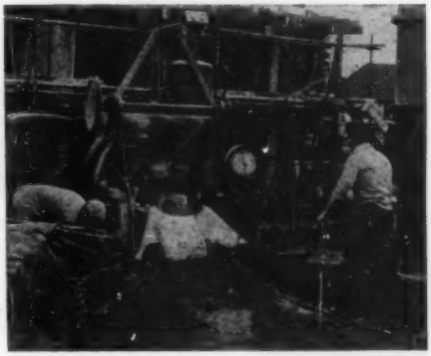

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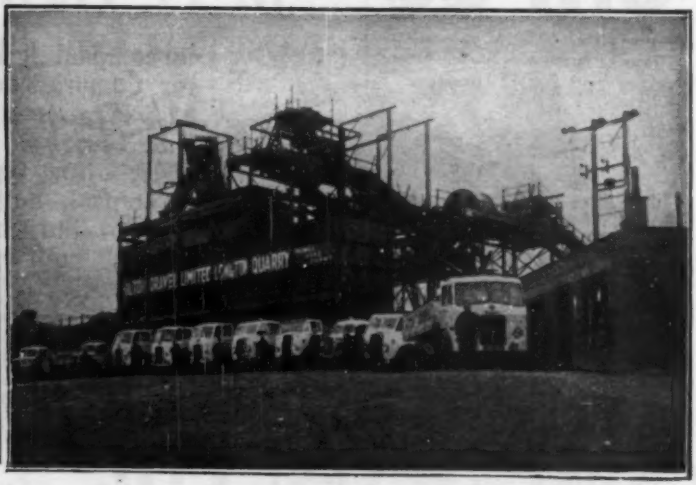

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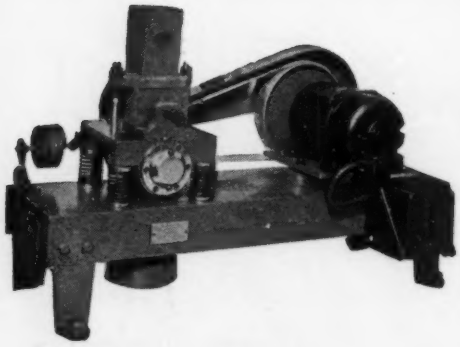

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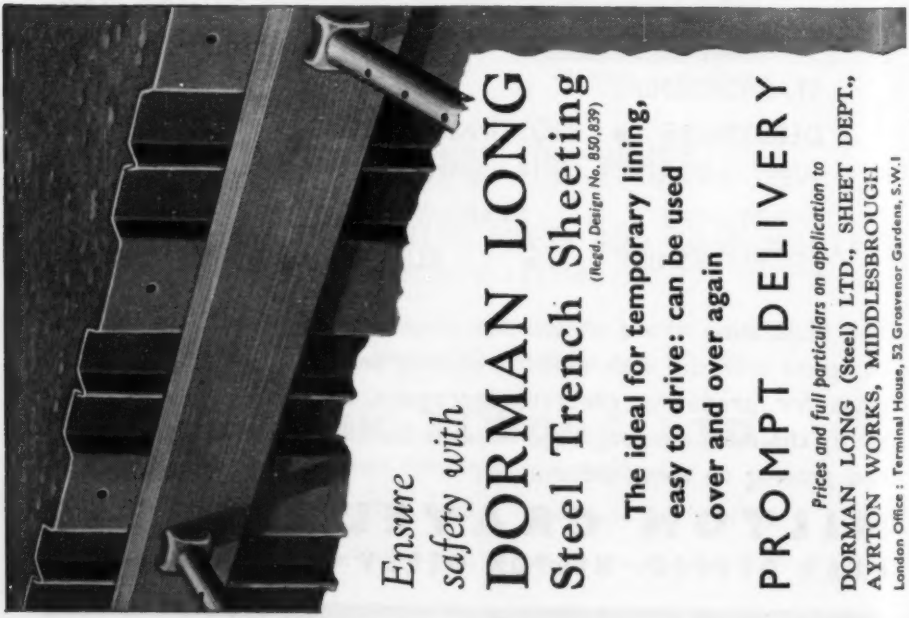

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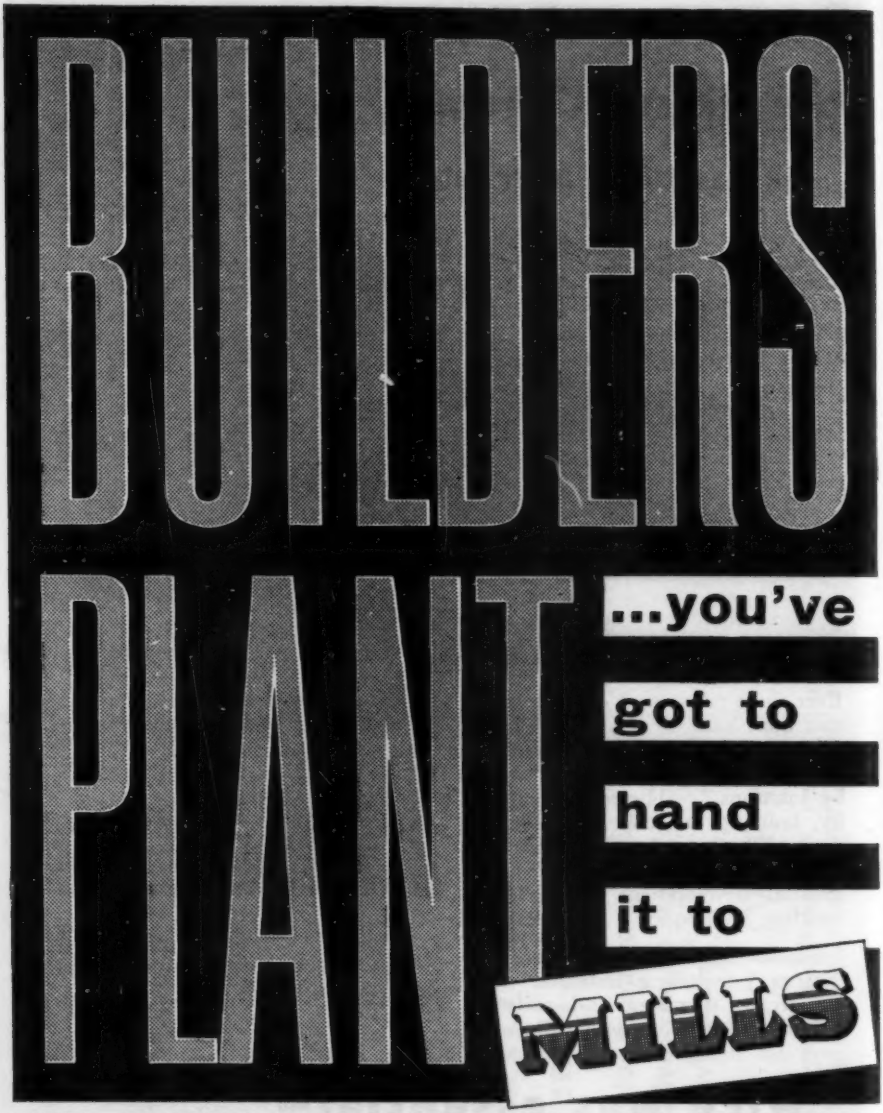

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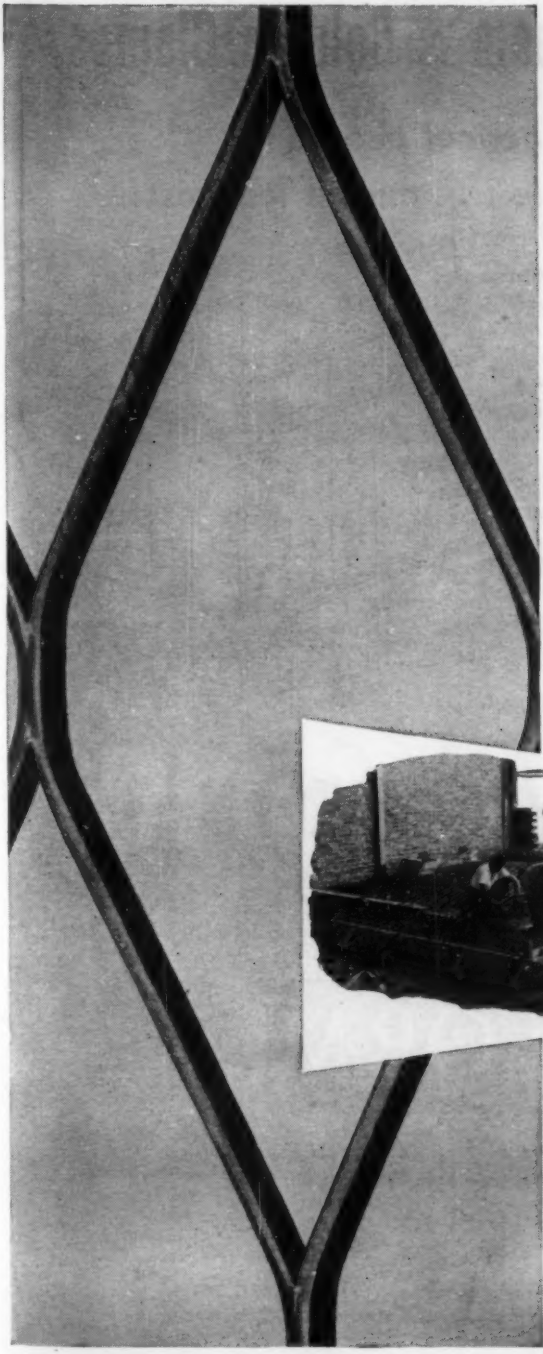

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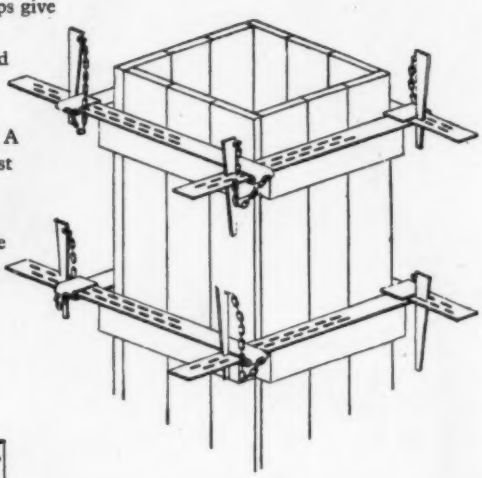

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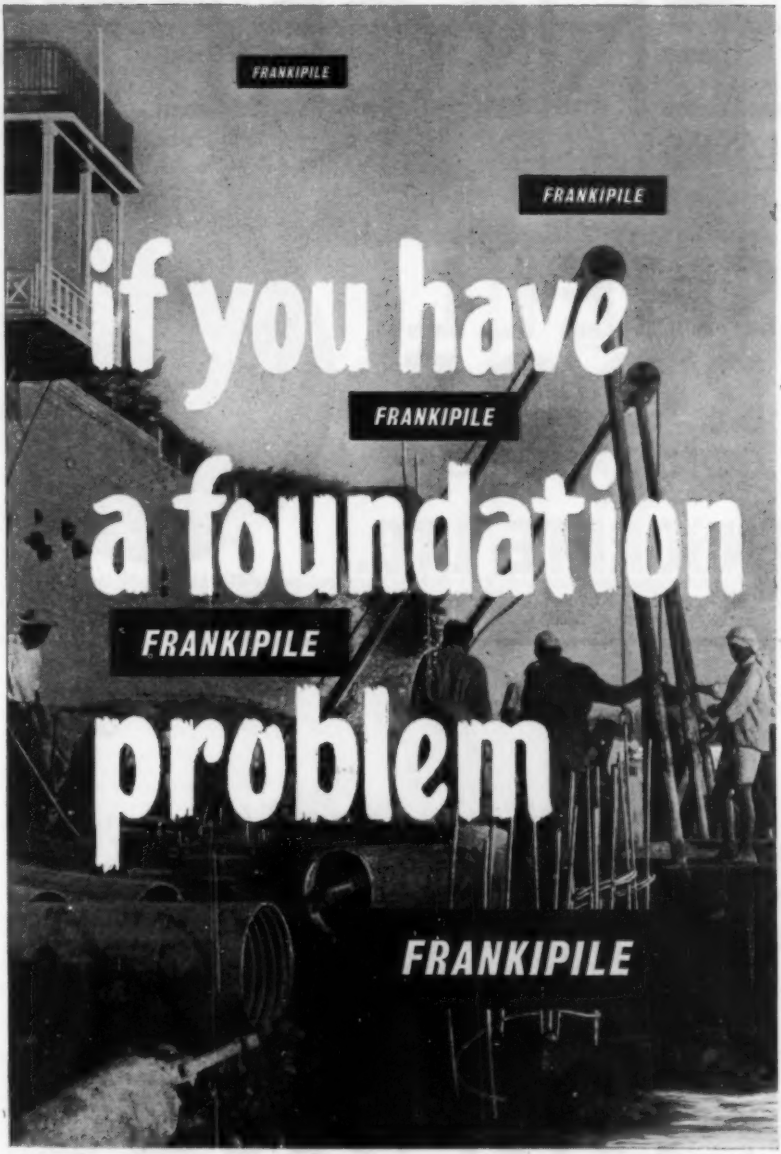

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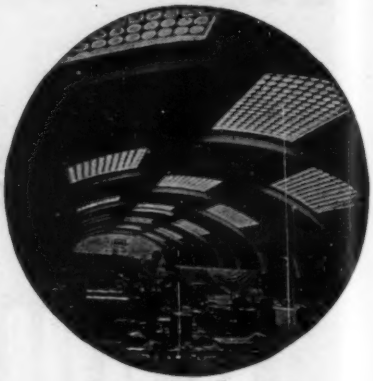

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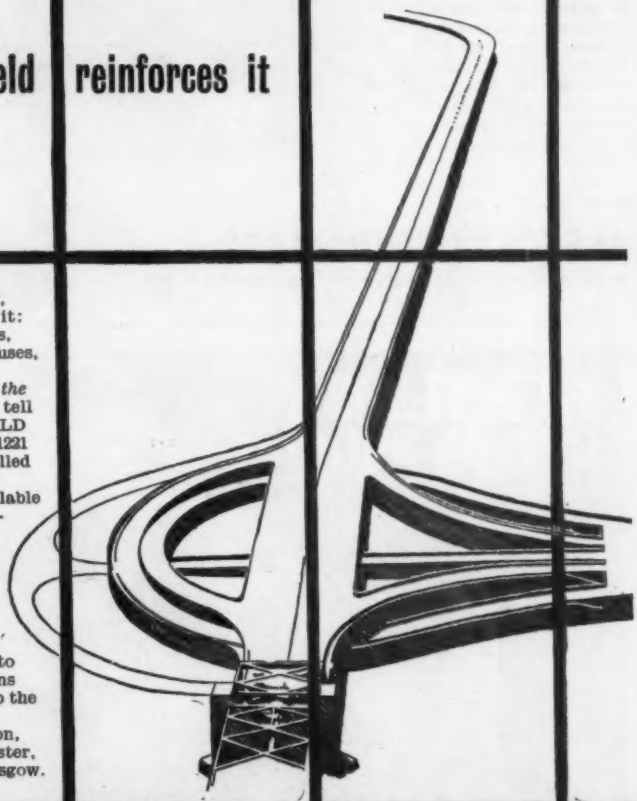

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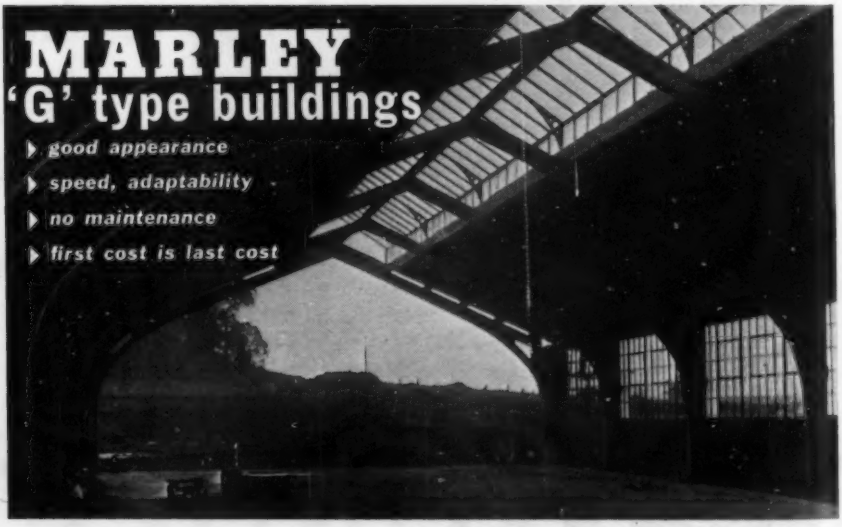

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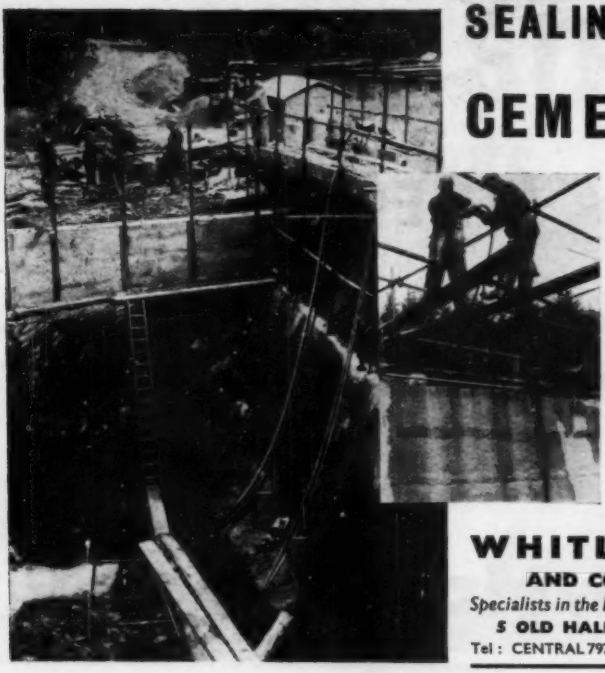

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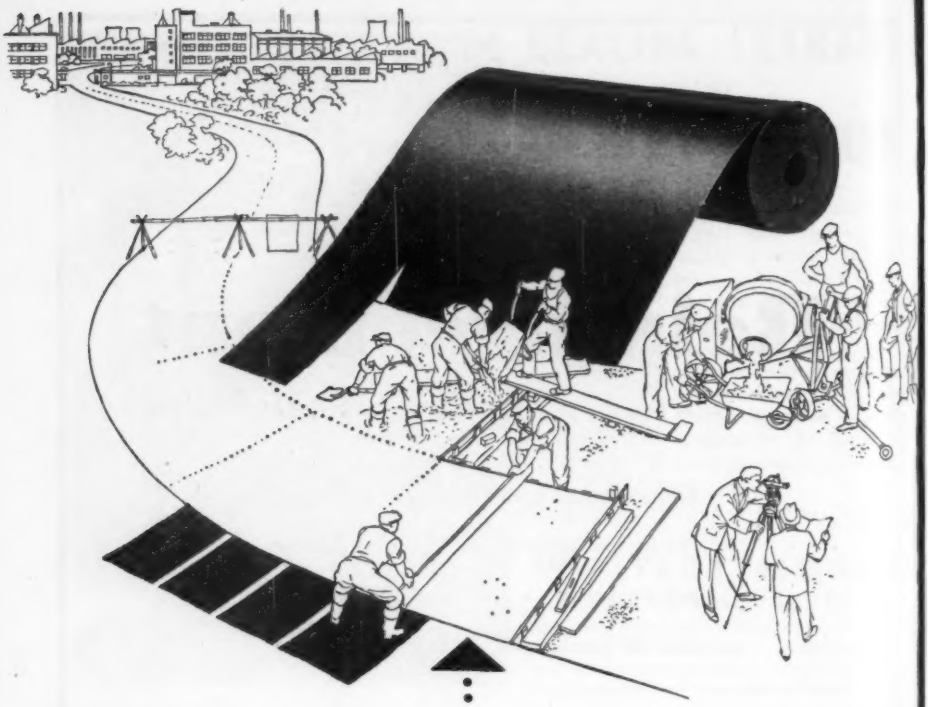

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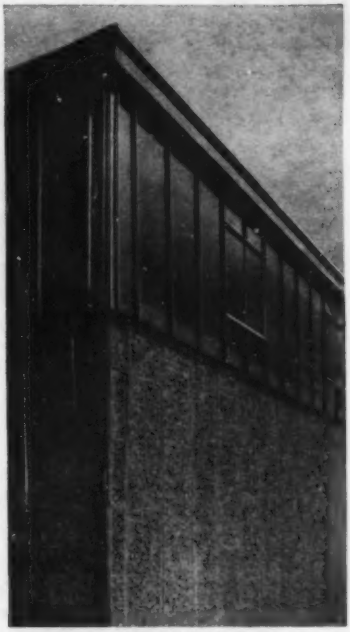

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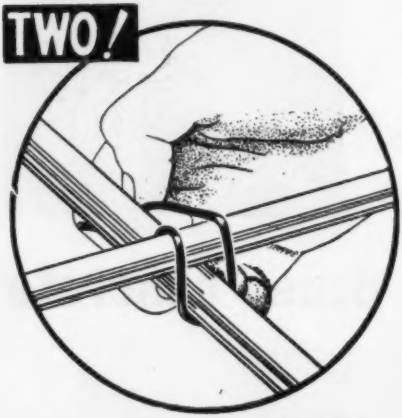

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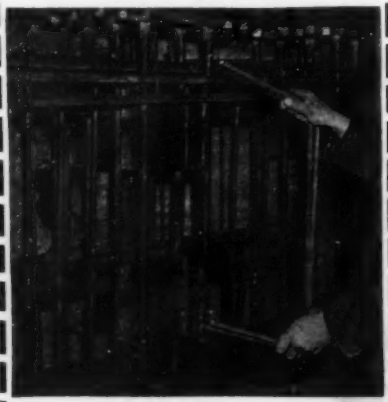

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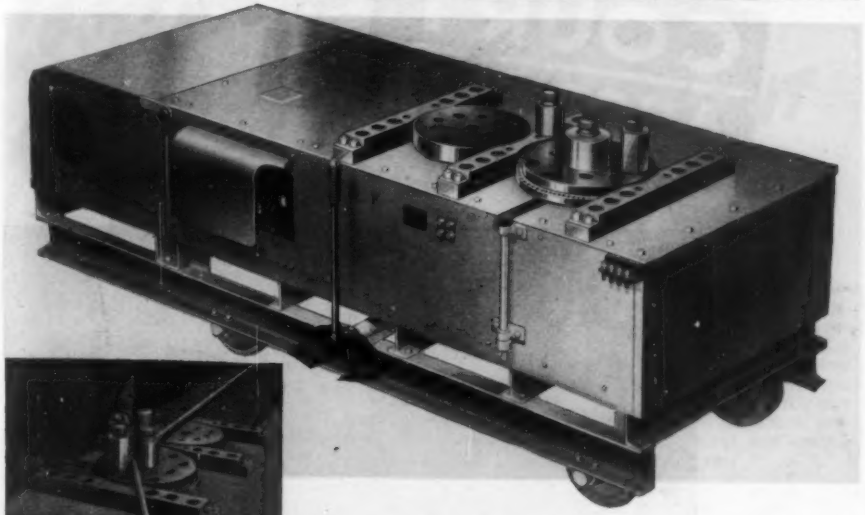

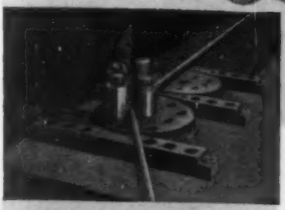

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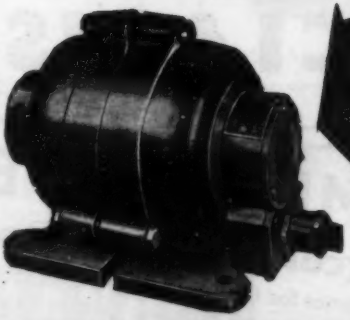

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume LII, No. 4. LONDON, APRIL, 1957.

EDITORIAL NOTES

The Responsibility for British Standards.

IN our February number we commented on a British Standard Glossary of Terms for Concrete and Reinforced Concrete sent to us by the British Standards Institution to be reviewed in this journal. In a critical comment we pointed out some terms that were wrongly defined and some examples of bad grammar. We expressed the opinion that the glossary was a very poor one that should not have been glorified with the title British Standard, and suggested that this example indicated that the Institution was not a suitable body to prepare glossaries. The following reply has been received from the Institution :

> " We have read with close interest the critical comment in your February issue on the new British Standard Glossary of Terms for Concrete and Reinforced Concrete (B.S. 2787). The fact is, of course, that in this field as in all others, B.S.I. is only the servant of informed industry, which uses the Institution's machinery to prepare the standards it requires. Attached, for your information, is a list of the important technical and industrial authorities who made up the committee which actually prepared B.S 2787. The chairman was ——, while other notable members included —— and ——. The glossary, after wide circulation of a preliminary draft, was also considered in final draft form by the Cement, Lime & Gypsum Products Industry Standards Committee of the B.S.I., whose membership is given on page 3 of the standard itself. When therefore you ask whether B.S.I. is a ' suitable body to prepare glossaries ' you are, in fact, questioning the authority and ability of the members of the industry who serve on these committees.
>
> " It is also worth mentioning that the present glossary is likely over a period of years to be considerably extended and to become a good deal more detailed in order effectively to cover the newer fields which are constantly being developed, such as prestressed concrete, to which you refer. With this fact in mind we can say that the committee responsible for the glossary will be very glad indeed to receive and to consider any constructive observations for its improvement which you feel able to submit—although in this connection it should be borne in mind that the glossary has been prepared much less for use by members of the concrete industry than for the guidance of the countless specifiers and users of its products such as architects and builders."

It is surprising that the Institution should repudiate responsibility for the contents of a document on which its name appears as the publisher, and which is stated in its foreword to be " published under the authority of the General Council ". It is more surprising that, in its anxiety to dissociate itself from an

unsigned document published under the authority of its General Council, the Institution should wish to publish the names of three only of the members of a committee as being responsible for this British Standard. As it is not stated that any of the members of the Committee disapproved of this document, it seems very unfair that the credit for it should be given to three only. For this reason we have omitted the names of the members of the committee, and the bodies they represented, from the foregoing letter from the Institution.

The statement in this letter that " in this field as in all others B.S.I. is only the servant of informed industry, which uses the Institution's machinery to prepare the standards it requires ", raises a point of considerable national importance. There has been much propaganda to persuade consumers that " British Standard " goods are of high quality, and people may be excused if they consider that goods so described are guaranteed by the Institution that issues them and that is heavily subsidised by the Government. Indeed, the Institution has registered a trade mark, known as the B.S.I. Kite Mark, which manufacturers are licensed to affix to products made in accordance with British Standards, but this mark indicates only that the Institution may ensure that the products comply with the standards prepared by the industry concerned. As we have pointed out in this journal, some of the British Standards for certain precast concrete goods require a poor quality only, while others prescribe no standard of quality at all. The words British Standards are the title of the Institution, and most people may be excused for assuming that goods so branded carry the hall-mark of the Institution. Yet it seems that the Institution makes no effort at all to scrutinise the value of the Standards it issues. The American Standards Association does not undertake the writing of standards, but it does examine standards prepared by trade associations and others and does not issue them without question, as the British Standards Institution states that it does.

It is true that those connected with industries to which British Standards apply are aware that the Institution is merely an organisation that arranges committees and publishes and sells the documents, but this is not generally known to the purchasers of the goods. It might be less misleading if these documents were called Manufacturers' Standards ; and if industries were to prepare these standards themselves (as the Institution states that they now do), and issue them themselves, there would be no need for a Government subsidy to maintain the British Standards Institution. It is a remarkable state of affairs that the Institution should issue documents under the authority of its General Council, and describe them as British Standards, without apparently accepting any responsibility for them or taking any trouble to ensure that the qualities prescribed in these documents are suitable for their purpose. The quality of the glossary in question is so poor that no responsible publisher would have issued it —yet it appears as a British Standard.

The Institution says it expects that the glossary will be used more by architects and builders than by people in the concrete industry ; surely this is a reason why more care should have been taken to make it accurate and grammatical, rather than an excuse for producing a poor document. It is also stated that constructive observations are desired. Some mistakes were mentioned in our February number, but the most constructive suggestion we can offer is that the present document be immediately withdrawn and a more worthy one issued.

Bending and Axial Stresses in Rectangular Columns.

By R. J. BARTLETT.

In calculating the stresses in a member subjected to axial load and a bending moment it is necessary, if tensile stresses are present, to solve a cubic equation in order to ascertain the value of the neutral axis or the neutral axis factor. If the member is rectangular with equal reinforcement in each side the equation is

$$n^3 + 3n^2\left(e - \frac{D}{2}\right) + \frac{6mAe}{B}n - \frac{3mA}{B}\left(eD + \frac{r^2}{2}\right) = 0 \quad . \quad . \quad (1)$$

If it has reinforcement in the tensile side only the equation is

$$n^3 - 3dn^2 - \frac{6m(Ws + M)}{Bt}n + \frac{6md(Ws + M)}{Bt} = 0 \quad . \quad . \quad (2)$$

[The derivation and notation are given in this journal for January 1955.]

The required value may be obtained with sufficient accuracy by substituting for this cubic equation equivalent quadratic equations each of which applies to a range of values of n. These equations may in turn be solved to produce formulæ expressing n directly.

In this method it is more convenient to consider n, r, d, and e in their ratios to D and pBD as the steel area in the first case, and n in its ratio to d in the second case. Equations (1) and (2) thus become (3) and (4) respectively.

$$n^3 + 3n^2(e - \tfrac{1}{2}) + 6mpen - 3mp\left(e + \frac{r^2}{2}\right) = 0 \quad . \quad . \quad (3)$$

$$n^3 - 3n^2 - \frac{6m(Ws + M)}{Btd^2}n + \frac{6m(Ws + M)}{Btd^2} = 0 \quad . \quad . \quad (4)$$

Equation (3) in any given case is then solved by one of the following formulæ.

Value of n.

0 to 0·2 $$n = \frac{-(K - 0{\cdot}004) - \sqrt{(K - 0{\cdot}004)^2 + L(e - 0{\cdot}4)}}{e - 0{\cdot}4} \quad . \quad . \quad (3a)$$

0·2 to 0·4 $$n = \frac{-(K - 0{\cdot}044) \mp \sqrt{(K - 0{\cdot}044)^2 + (L - 0{\cdot}008)(e - 0{\cdot}2)}}{e - 0{\cdot}2} \quad (3b)$$

0·4 to 0·6 $$n = \frac{-(K - 0{\cdot}124) + \sqrt{(K - 0{\cdot}124)^2 + (L - 0{\cdot}04)e}}{e} \quad . \quad . \quad (3c)$$

0·6 to 0·8 $$n = \frac{-(K - 0{\cdot}244) + \sqrt{(K - 0{\cdot}244)^2 + (L - 0{\cdot}112)(e + 0{\cdot}2)}}{e + 0{\cdot}2} \quad (3d)$$

0·8 to 1·0 $$n = \frac{-(K - 0{\cdot}404) + \sqrt{(K - 0{\cdot}404)^2 + (L - 0{\cdot}24)(e + 0{\cdot}4)}}{e + 0{\cdot}4} \quad (3e)$$

where $K = epm$ and $L = \dfrac{pmr^2}{2} + epm$.

Then $$c = \frac{2W}{BD\left(n + pm\dfrac{2n - 1}{n}\right)} \quad \text{and} \quad t = \frac{mc(1 - n)}{n}.$$

If $(m - 1)$ is used instead of m in relation to the reinforcement in compression, add $\frac{p(r - 2e)}{4}$ to K and $\frac{p(1 - r)(r - 2e)}{4}$ to L, and subtract $\frac{p(2n - 1 + r)}{2n}$ from the expression in brackets in the formula for c.

Also equation (4) is solved by the following formulæ, where

$$K = \frac{m(Ws + M)}{Btd^2}.$$

Value of n.

0 to 0·2	$n = -(0{\cdot}004 + 1{\cdot}111k) + \sqrt{2{\cdot}233K + 1{\cdot}235K^2}$ . . .	(4a)
0·2 to 0·4	$n = -(0{\cdot}062 + 1{\cdot}429K) + \sqrt{0{\cdot}015 + 3{\cdot}035K + 2{\cdot}042K^2}$.	(4b)
0·4 to 0·6	$n = -(0{\cdot}247 + 2{\cdot}0K) + \sqrt{0{\cdot}141 + 4{\cdot}988K + 4{\cdot}0K^2}$ . .	(4c)
0·6 to 0·8	$n = -(0{\cdot}811 + 3{\cdot}333K) + \sqrt{1{\cdot}031 + 12{\cdot}08K + 11{\cdot}111K^2}$.	(4d)
0·8 to 1·0	$n = -(4{\cdot}035 + 10{\cdot}0K) + \sqrt{18{\cdot}70 + 100{\cdot}70K + 100{\cdot}0K^2}$.	(4e)

Then $c = \frac{tn}{m(1 - n)}$, and the area of steel (A) is $\frac{cBnd - 2W}{2t}$.

In any problem it must be decided which of the five equations to use. The choice is narrow since a and b of each set generally apply only when the axial load is tensile, c when the ratio of bending moment to axial load is large, and the remainder when this ratio becomes comparable with the depth of the section itself, but not less than about one-sixth of this depth. The correct formula must be used to obtain an accurate result.

Examples.

(1) A column 12 in. by 9 in. in cross section and reinforced with two $\frac{7}{8}$-in. bars in both the compressive and tensile sides, is subjected to a direct load of 22,500 lb. and a bending moment of 150,000 in.-lb. Calculate the maximum stresses if $rD = 9$ in., $dD = 10{\cdot}5$ in., and $m = 15$. (*Fig.* 1.)

The eccentricity factor e is $\frac{150{,}000}{22{,}500 \times 12} = 0{\cdot}556$.

$$p = \frac{2{\cdot}41}{12 \times 9} = 0{\cdot}0223. \quad K = 15 \times 0{\cdot}0223 \times 0{\cdot}556 = 0{\cdot}186.$$

$$L = \frac{0{\cdot}0223 \times 15 \times 0{\cdot}75^2}{2} + 0{\cdot}186 = 0{\cdot}094 + 0{\cdot}186 = 0{\cdot}280.$$

Using formula (3c), $n = 0{\cdot}556$. Therefore

$$c = \frac{45{,}000}{108\left(0{\cdot}556 + \frac{0{\cdot}3345 \times 0{\cdot}112}{0{\cdot}556}\right)} = 668 \text{ lb. per square inch.}$$

$t = 5750$ lb. per square inch.

(2) As example (1) but with the direct load reversed to become a tensile load. The eccentricity factor e is $-0{\cdot}556$. $p = 0{\cdot}0223$. $K = -0{\cdot}186$. $L = 0{\cdot}094 - 0{\cdot}186 = -0{\cdot}092$. (*Fig.* 1.)

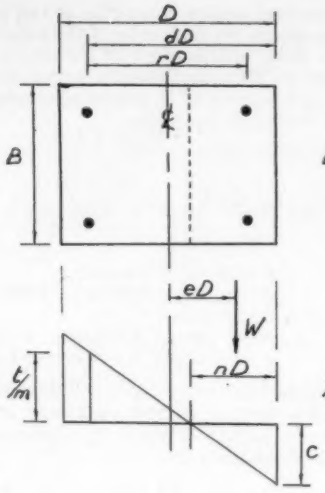


Fig. 1.

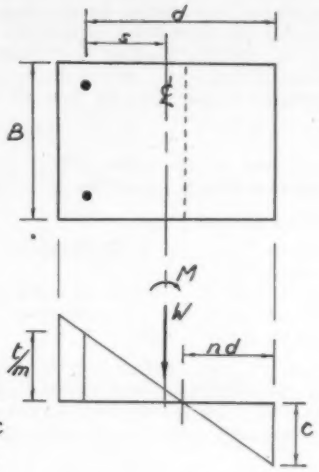


Fig. 2.

Using formula (3a), $n = 0{\cdot}170$.

$$c = \frac{-45{,}000}{108\left(0{\cdot}17 + 0{\cdot}3345\dfrac{0{\cdot}34 - 1{\cdot}0}{0{\cdot}17}\right)} = 370 \text{ lb. per square inch.}$$

Therefore $t = \dfrac{15 \times 370 \times 0{\cdot}705}{0{\cdot}17} = 23{,}000$ lb. per square inch.

(3) A member 12 in. by 9 in. in cross section is subjected to a thrust W of 22,500 lb. and a bending moment M of 150,000 in.-lb. If t is 18,000 lb. per square inch, d is 10·5 in., s is 4·5 in., and m is 15, calculate c and A (*Fig.* 2).

$$M + Ws = 251{,}250 \text{ in.-lb.} \quad K = \frac{15 \times 251{,}250}{9 \times 18{,}000 \times 10{\cdot}5^2} = 0{\cdot}2105.$$

Using formula (4c),

$$n = -(0{\cdot}247 + 2 \times 0{\cdot}2105) + \sqrt{0{\cdot}141 + 4{\cdot}988 \times 0{\cdot}2105 + 4 \times 0{\cdot}2105^2} = 0{\cdot}502.$$

Therefore $$c = \frac{18{,}000 \times 0{\cdot}502}{15 \times 0{\cdot}498} = 1210 \text{ lb. per square inch.}$$

$$A = \frac{57{,}350 - 45{,}000}{36{,}000} = 0{\cdot}343 \text{ sq. in.}$$

NOTES.

Equation (3) may be rearranged as in (5).

$$en^2 + 2mpen - mp\left(e + \frac{r^2}{2}\right) = n^2\left(\frac{1}{2} - \frac{n}{3}\right) \quad . \quad . \quad . \quad (5)$$

The solution of this equation may be considered as the value of n at the intersection of the two curves represented by the expressions on each side of the equation. The curve given by the right-hand expression, being independent of the properties of the section and the external loading, is constant for all cases to which equation (3) applies. The ordinates of this curve, shown in *Fig.* 3 by curve ACB, are, in effect, the addition of the ordinates of the straight line AB with the equation

$$y = 0{\cdot}167n \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

and the ordinates of the curve AED.

For various ranges of values of n this curve coincides with other curves:

Values of n.	Curve.	
0 to 0·2	The parabola	$y = 0{\cdot}4n^2 - 0{\cdot}160n.$
0·2 to 0·4	,,	$y = 0{\cdot}2n^2 - 0{\cdot}080n - 0{\cdot}008.$
0·4 to 0·6	The straight line	$y = 0{\cdot}08n.$
0·6 to 0·8	The parabola	$y = -0{\cdot}2n^2 + 0{\cdot}32n - 0{\cdot}112.$
0·8 to 1·0	,,	$y = -0{\cdot}4n^2 + 0{\cdot}64n - 0{\cdot}24.$

If equation (6) is added to each of these and the resulting expressions substituted in turn for the right-hand expression in (5), then quadratic equations are formed. Formulæ (3 a to e) are the solutions of these equations obtained by algebraic formula.

In like manner, equation (4) may be rearranged as

$$-\frac{n^2}{2} - 2Kn + 2K = n^2\left(\frac{1}{2} - \frac{n}{3}\right),$$

and, by using the foregoing procedure, formulæ (4a to e) are similarly derived.

Unequal Tensile and Compressive Reinforcement.

The use of the foregoing formulæ may be extended to the stress-analysis of reinforced concrete sections in which the amounts of reinforcement in each face are unequal. In such cases the expressions for K, L, and c acquire extra terms:

$$K = epm + \frac{rqm}{2}. \quad L = \frac{pmr^2}{2} + epm + rqm(e + \tfrac{1}{2}),$$

$$c = \frac{2W}{\cdot\, BD\left[n + pm\dfrac{(2n-1)}{n} - \dfrac{rqm}{n}\right]},$$

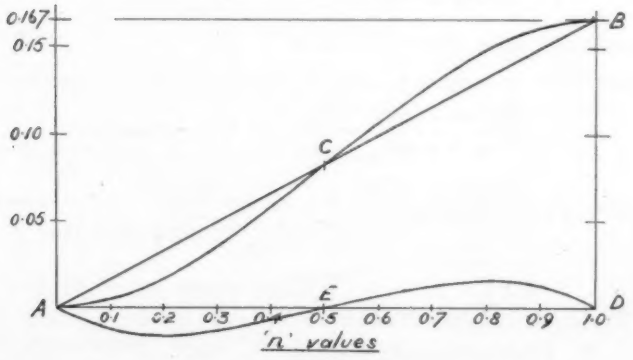


Fig. 3.

where q is the difference between the ratios of tensile and compressive reinforcement, while p remains the ratio of the total steel used.

EXAMPLE.—As the first example but with two $\frac{7}{16}$-in. diameter bars only in the compressive face.

$$p = \frac{1{\cdot}203 + 0{\cdot}301}{108} = 0{\cdot}0139. \quad q = \frac{1{\cdot}203 - 0{\cdot}301}{108} = 0{\cdot}00835.$$

$$K = 0{\cdot}556 \times 0{\cdot}0139 \times 15 + \frac{0{\cdot}75 \times 0{\cdot}00835 \times 15}{2} = 0{\cdot}1161 + 0{\cdot}0469 = 0{\cdot}1630.$$

$$L = \frac{0{\cdot}0139 \times 15 \times 0{\cdot}75^2}{2} + 0{\cdot}1161 + 0{\cdot}0938 \times 1{\cdot}056$$

$$= 0{\cdot}0587 + 0{\cdot}1161 + 0{\cdot}0990 = 0{\cdot}2738.$$

Using formula (3c),

$$n = \frac{-(0{\cdot}1630 - 0{\cdot}1240) + \sqrt{0{\cdot}0390^2 + 0{\cdot}2338 \times 0{\cdot}556}}{0{\cdot}556} = 0{\cdot}5810.$$

Then

$$c = \frac{45{,}000}{108\left[0{\cdot}581 + \frac{0{\cdot}209 \times 0{\cdot}164}{0{\cdot}581} - \frac{0{\cdot}0938}{0{\cdot}581}\right]} = 872 \text{ lb. per square inch}$$

and $$t = \frac{872 \times 15 \times 0{\cdot}419}{0{\cdot}581} = 9440 \text{ lb. per square inch.}$$

These adjusted expressions are obtained by deriving a new equation (3) in a similar manner as before but reducing the total compressive force in the column section by $qBDmc\dfrac{\left(n - \frac{1}{2} + \frac{r}{2}\right)}{n}$ to account for the effect of the ratio of compressive reinforcement now being reduced by an amount q.

It is assumed that both groups of bars are equidistant from the centre of the section. This, however, will be untrue if the bars have equal concrete cover. The error is insignificant in many cases, but it may be avoided by reducing r in the terms containing rqm in the expressions for K, L and c by an amount equal to $\dfrac{2xy}{D(1-x)}$, where x is the ratio of the areas of compressive to tensile reinforcement, and y is the difference between the distances of the centres of gravity of each group of bars and its respective section face. This expression is derived from the geometry of the stress distribution diagram and by substituting

$$qBDmc\frac{\left(n - \frac{1}{2} + \frac{r}{2} - \frac{y}{D}\right)}{n}$$

for the expression in the foregoing paragraph in deriving the new equation (3).

This process, however, applied to examples such as the one worked produces only insignificant differences in the stresses already obtained.

NOTE.—These formulæ give solutions to the original cubic equations correct to three decimal places and are obtainable by slide-rule. The values of n calculated in the examples given in the January 1955 number would require a second adjustment in order to obtain the same degree of accuracy as that given by the foregoing method.

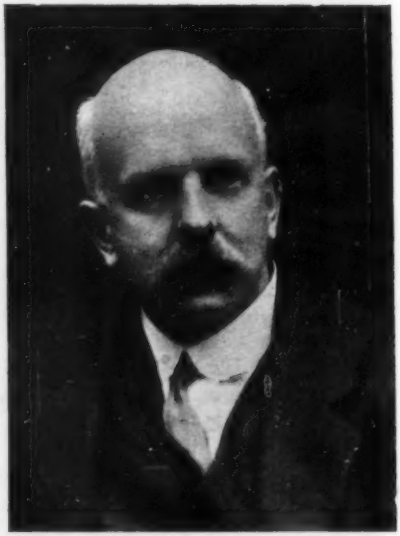
The Late Mr. R. W. Vawdrey.

By the death of R. W. Vawdrey on February 24, at his home at Limpsfield in Surrey, at the age of 82, we have lost one of the last survivors of the leaders in the early struggle to win for reinforced concrete the position it now holds as a reliable and economical structural material.

After leaving King Edward's School, Birmingham, Vawdrey graduated at Pembroke College, Cambridge, and his keen interest in mathematics led him to choose civil engineering as a profession. As a pupil with Messrs. Middleton, Hunter & Duff, and later with Messrs. Wilcox & Raikes, he gained practical experience in the use of concrete on such works as a drainage scheme at Pennicuik, the great reservoirs at Staines, and water-works at Kidderminster. As a result of this experience, and having been elected an Associate Member of the Institution of Civil Engineers in 1902, in 1905 he contributed to the Institution a paper on the grouting of prepacked aggregate.

It was no doubt some knowledge of this previous experience in the use of concrete that in 1906 led the late Sir William Mather, P.C., to choose him as chief engineer of the newly-formed Patent Indented Steel Bar Co. (now the Indented Bar & Concrete Engineering Co., Ltd.), of which he later became managing director and ultimately chairman.

In those early days of intense competition with other systems, and when the regulations controlling the use of reinforced concrete might be said to be flexible, success depended on the adequate yet kindly guidance given by Vawdrey to new members of the staff (including the writer) while leaving them sufficient freedom to feel a personal responsibility for their designs. As a result there are throughout the country many eminent engineers who at one time or another have profited by serving under him, and at least three of them have become Presidents of the Institution of Structural Engineers of which he was a founder Member.



The late R. W. Vawdrey.

To have controlled a business successfully for half a century, including two wars, and in that time to have been responsible for such an enormous amount of useful work, is an achievement of which anyone may well be satisfied. But this does not alter the loss his passing means to his wife and family and the many friends who had known him for so long.

P. G. BOWIE.

Prestressed Concrete in the Netherlands.

THE following is an abstract of a lecture given in London in January last by Mr. A. S. G. Bruggeling under the auspices of the Precast Concrete Development Group of the Cement and Concrete Association.

Fire Resistance.

Tests on the fire-resistance of prestressed beams showed that the fire-resistance was 51 minutes when the cover of concrete was 0·8 in., 62 minutes with 1·2 in. cover, and 83 minutes with 2 in. cover. Beams with post-tensioned grouted steel had ten minutes' longer

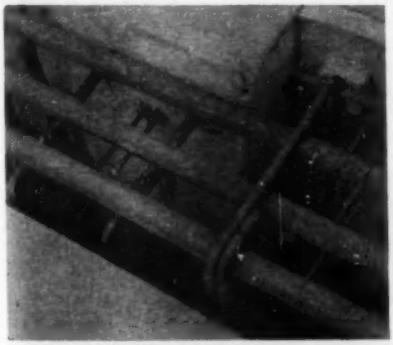


Fig. 1.—Prestressed Beams Connected to Edge-beams by Mild Steel Reinforcement.

resistance to fire than beams with pre-tensioned wire, probably because the grout provides a thermal insulation to the cables. It may be assumed that the joints in beams made of separate blocks may decrease the fire-resistance because the joints may disintegrate at an early stage. Since a fire-resistance of at least two hours is required for buildings in the Netherlands, the use of prestressed concrete is excluded, unless fireproof cover be provided. This is considered as discriminating against prestressed concrete, because all the tests were carried out on freely-supported beams. In the case of multiple-story buildings, beams built-in at the ends are more acceptable since the floor usually prevents high temperatures in the beams. It is generally supposed that a statically-indeterminate structure has increased resistance to fire; it may also be assumed that stirrups in prestressed concrete improve its resistance to fire.

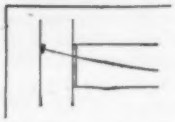


Fig. 2.—Beam Prestressed to Column.

Efficiency of Grouting.

If the grouting is not properly done when metal sheaths are used, there is a possibility that pockets of water may occur in the sheaths with the consequent risk of damage if this water freezes. The addition of spirit to the grout lowers the freezing point and thus diminishes the frost danger; the quality of the grout is not noticeably reduced but the hardening of the mortar is considerably retarded; it is possible, however, to reduce the hardening time by the use of calcium chloride in the grout.

In the case of a roof at Emmen, built in 1951–4, covering a total floor area of 280,000 sq. ft., the slabs were placed on the upper flanges of the beams and the ceiling slabs on the bottom flanges; the ducts formed were used for air-conditioning.

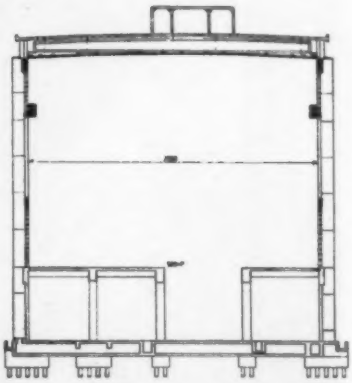


Fig. 3.—Cross Section of Building shown in Fig. 4.

Joining Beams over Supports.

Because of the elongation at the supports of simply-supported beams, a method of improving the continuity is the use of mild-steel reinforcement projecting from the prestressed beams (*Fig.* 1) and incorporated in prestressed edge-beams. A disadvantage is that the floor beams have to be in position before the shuttering for the edge-beams is erected. A joint between beams and columns has recently been developed by the application of complete prestressing with straight or nearly-straight cables (*Fig.* 2).

A paper factory at Wormer is 405 ft. long, 63 ft. high, and 66 ft. wide (*Figs.* 3 and 4). The frame comprises columns, gable-beams, crane-beams, eaves-beams, roof beams, and ceiling, and these were formed of units joined by prestressing after assembly. The columns comprise eight blocks 1 ft. 8 in. by 3 ft. 10 in. in cross section. They have a central hole to accommodate four cables of twelve 7-mm. wires. The joints are 0·4 in. wide. The columns were assembled inside a steel lattice erected by means of a crane; the blocks were placed within it during one day, and the next day the cables were inserted, tensioned, and grouted.

The cables were fixed in the foundation so that the column could resist wind pressure independently. There is an expansion joint every four or five bays with double columns. The gable-beams rest on steel angles fixed to the columns, and are prestressed from end to end. The roof-beams were positioned on centering travelling on the crane-beams. During the first day the centering and the blocks were erected and the joints (about $\frac{3}{4}$ in. wide) were mortared, and during the next day the cables were inserted and half of them were tensioned. The ceiling slabs rest on the lower flanges of the roof trusses. The slabs are 2 in. thick by 10 ft. square, with strengthening ribs, and prestressed, from expansion joint to expansion joint, by cables of 5-mm. wires. After these cables were tensioned, the

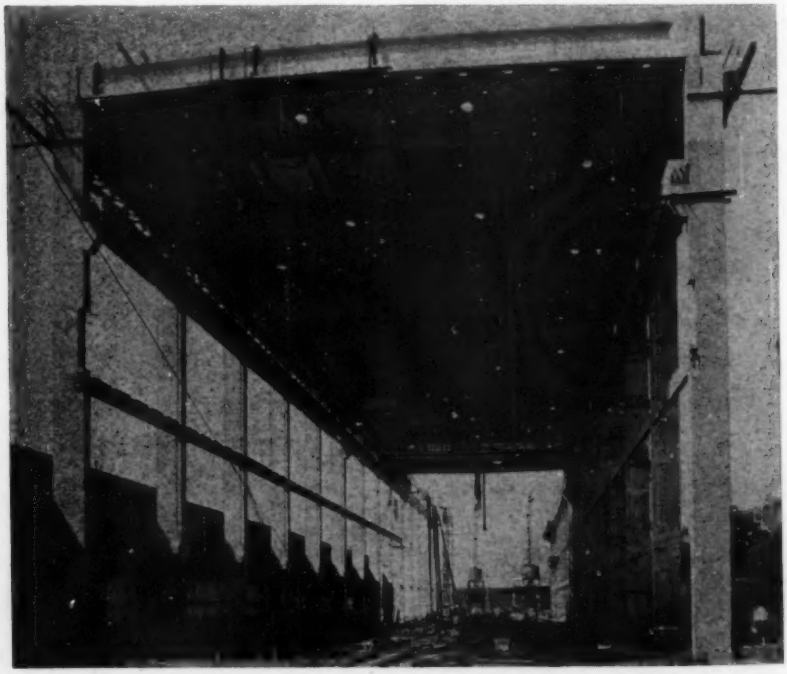


Fig. 4.—A Prestressed Frame (*see also Fig.* 3).

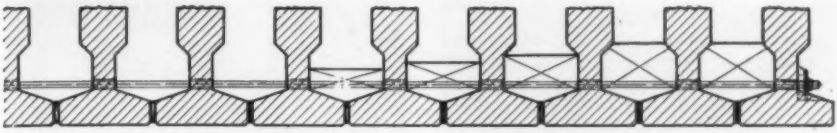


Fig. 5.—Bridge Deck Prestressed Transversely.

remainder of the cables in the roof-beams were tensioned and grouted. The friction between the flange of the beam and the ceiling slab, created by prestressing in the longitudinal direction, is such that the ceiling slabs are also prestressed transversely.

Especially with factory-made units, the need was felt for a continuity bar to connect the beams at the supports. A common type of bridge comprises prestressed precast beams combined with a concrete slab cast in place. The beams may form a grille or a continuous soffit (*Fig.* 5). In bridges of this type it is possible to increase the height at the supports, and this extra height is used for placing continuity bars which serve to reduce tensile stresses due to moments at the supports (*Fig.* 6).

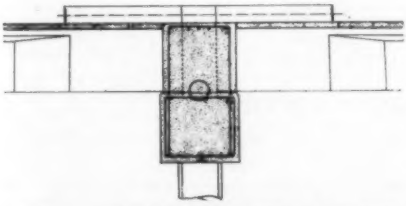


Fig. 6.—Prestressing over a Support (with Secondary Moments).

Two possible methods of carrying out the work are shown in *Fig.* 7. (a) The joints at the supports may be filled throughout their whole depth, in which case secondary stresses will be created by tensioning the continuity bars; (b) The joints at the supports may be only partly filled, in which case secondary stresses are almost avoided. In the latter case the amount of prestress required to ensure continuity is smaller than in the former, though casting the concrete near the supports is more difficult. An example of case (a) is a pier at Amsterdam with a grille of beams (*Fig.* 8); an example of case (b) is a slab highway bridge (*Fig.* 9).

When the beams are connected by transverse prestressing, secondary moments do not occur. If the ends of the beams extend over the supports, instead of making butt joints between the ends of the beams they may overlap transversely (*Fig.* 10). By forming mortar

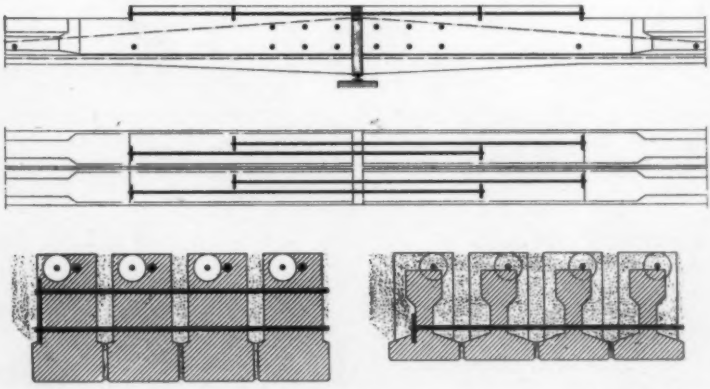


Fig. 7.—Prestressing over a Support (without Secondary Moments).

Fig. 8.—Prestressing over a Support.

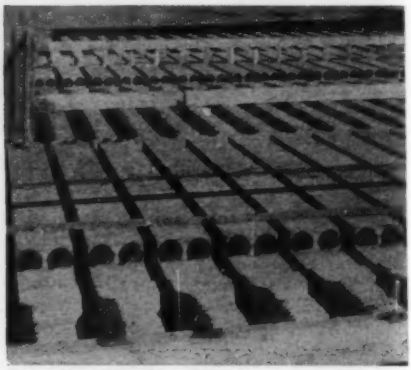


Fig. 9.—Prestressing over a Support.

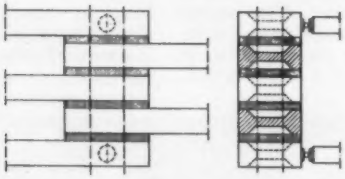


Fig. 10.—Connecting Beams over a Support.

Fig. 11.—Connecting Beams over a Support.

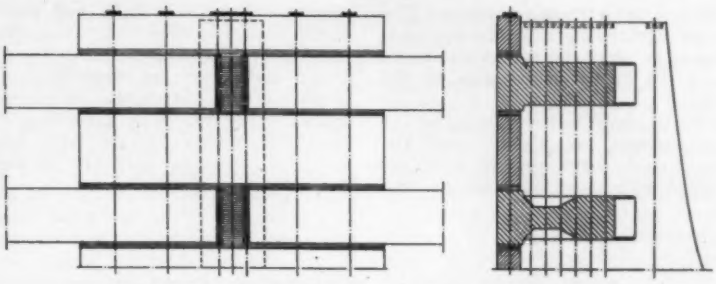


Fig. 12.—Connecting Beams over a Support.

joints between the overlapping ends and prestressing them transversely the ends of the beams are firmly bonded together. Moreover, over the supports a rigid transverse beam is formed which also transfers the load to the supports. A disadvantage of this method is that the outer face of the beams is not continuous. If the beams do not meet over the supports they may be connected by tee-beams whose stems lie between the beams and which join the beams over the supports; the assembly is then prestressed transversely (*Fig.* 11).

If the beams are close together they have a narrower upper flange near the supports than in the frame, and the continuity members are placed in the space so formed. The joints between these members and the main beams are filled with concrete, after which the transverse prestress is applied; the transverse cables pass through the continuity members (*Figs.* 12 and 13).

In slab bridges the connecting units are also placed between the upper sides of the beams, and often project above the beams, being incorporated in the concrete cover. These connecting beams are provided with ducts (*Figs.* 14 and 15) to accommodate the transverse prestressing cables.

Where the beams end near the point of zero bending moment they are connected to each other by elements fixed between the beams, and transversely prestressed rigid connections are obtained at the joint (*Fig.* 17).

Continuity in slab bridges has been provided in several instances by mild steel reinforcement. Reinforcement projects from the beams near the supports, and this, with additional mild steel reinforcement if necessary, is then concreted.

The system of longitudinal prestressing producing secondary moments usually requires additional points of support. This also applies to the methods of transverse prestressing. With the other methods it is possible to build a bridge without staging by temporarily supporting the beams. Staging is not required if the beams or the slab rest directly on

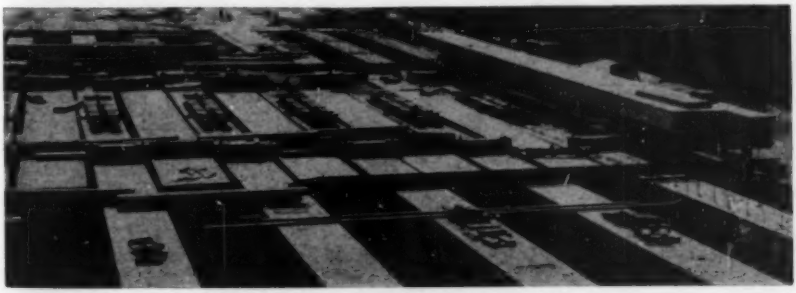


Fig. 13.—Assembling Connecting Beams.

the piers, without a transverse beam. The transverse prestressing near the supports is increased so that the transverse beam can resist the direct distribution of the prestress.

In order to simplify the placing of the connecting beams, and also to avoid the grouting of cable ducts, precast units are often suspended between the beams. The joint between these units and the webs of the beams is filled and, when this has hardened sufficiently, the bars placed above the units are tensioned. Overturning of the edge-beams and adjoining beams is prevented by providing raised connecting beams (*Fig.* 16). After tensioning, the cables or bars are encased in concrete.

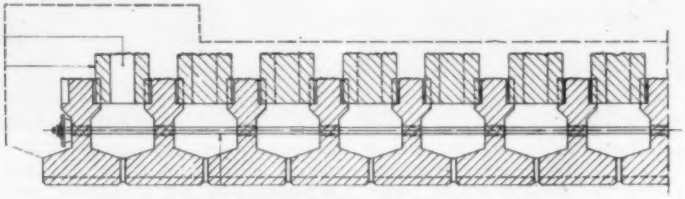


Fig. 14.—Connecting Beams over a Support.

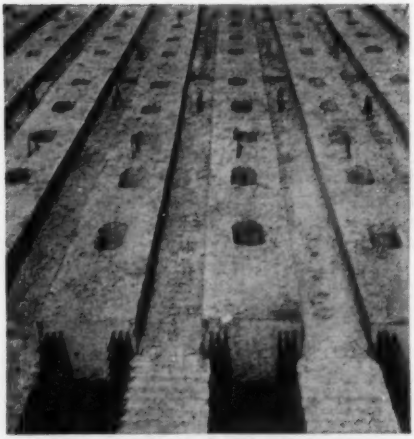


Fig. 15.—Connecting Beams where there is no Bending Moment.

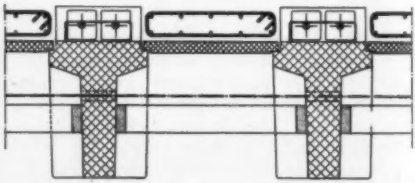


Fig. 16.—Prestressed Beams and Slabs.

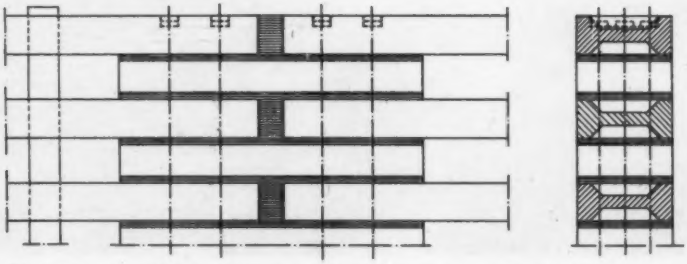


Fig. 17.—Connecting Beams where there is no Bending Moment.

An Unusual Staircase.

In order that the floor area should be completely free from obstruction, a staircase at the premises of the Times Furnishing Co., Ltd., Southchurch Road, Southend-on-Sea, was designed with supports at the floors only. The general arrangement is shown in *Fig.* 2(a). The staircase was designed for an imposed load of 100 lb. per square foot; the total dead load per square foot of the projected area of the steps is 159 lb., and of the half-landings 63 lb. per square foot (average).

The concrete was a nominal 1 : 1 : 2 mixture with a minimum compressive strength of 6000 lb. per square inch at 28 days. Square twisted high-tensile bars with a minimum yield-point of 75,000 lb. per square inch were used. The working stresses in the concrete were 1500 lb. per square inch compression in bending and 150 lb. per square inch in shear; the working tensile stress in the steel was 27,000 lb. per square inch.

To facilitate detailing the reinforcement, the following method of design was adopted. (1) The flights and half-landings between floors were considered as V-shape frames in a vertical plane under all possible combinations of fixation and loading, the worst cases being determined by superposition. Assuming an "ideal" frame as shown in elevation in *Fig.* 2(b), there are three superloads, namely either of the flights and half a landing. Points A and C were considered either as fixed or hinged according to which conditions produced the greatest moments in the other parts of the frame. (2) In plan, each unit was considered as a rectangular frame, fixed at A and B as in *Fig.* 2(c), which tends to rotate in the direction of the arrow under the influence of the moment $R_e X$ due to the reactions R_a and R_e at the top and bottom respectively.

In the calculations for the bending moments, the torque was considered to be

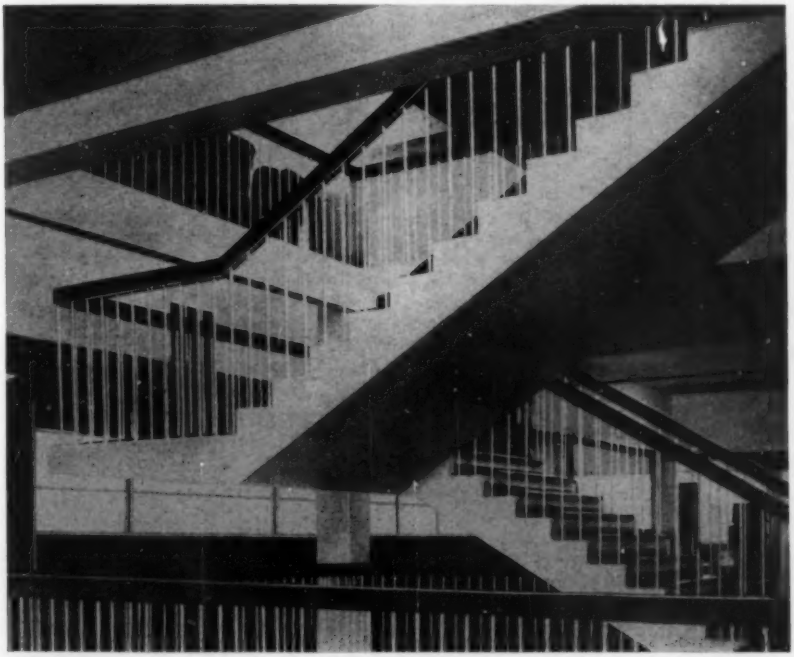


Fig. 1.

equivalent to that caused by a force F, equal to $\frac{R_a X}{Y}$, acting in the horizontal plane at B on the rectangular frame ABC. As the stairs and landings are not in the same plane, either horizontally or vertically, torsional moments are produced at B on the horizontal axis in case (1), and in the steps, in the plane perpendicular to the plane of the steps, in case (2).

The maximum positive moment on the stairs occurs when A and C are hinged and there is no live load on the landing but flights AB and BC are both loaded. In these circumstances B acts as a perfect hinge and the positive moments are modified only by the cantilever moment at B due to the dead load.

The moments in the horizontal plane at the main landings A and C, due to the thrust and pull of the stairs, are resisted by a solid section of floor, 7½ in. thick by 3 ft. 10⅝ in. wide, which forms a beam transmitting the forces to the steel frame. The fixing moments at A and C were provided by torsion in the composite beam at the edge of each landing. This beam was composed of a steel joist, strengthened with reinforcement bars and binding in a concrete casing to enable it to resist torsion, vertical shearing, and bending.

The calculations were simple but

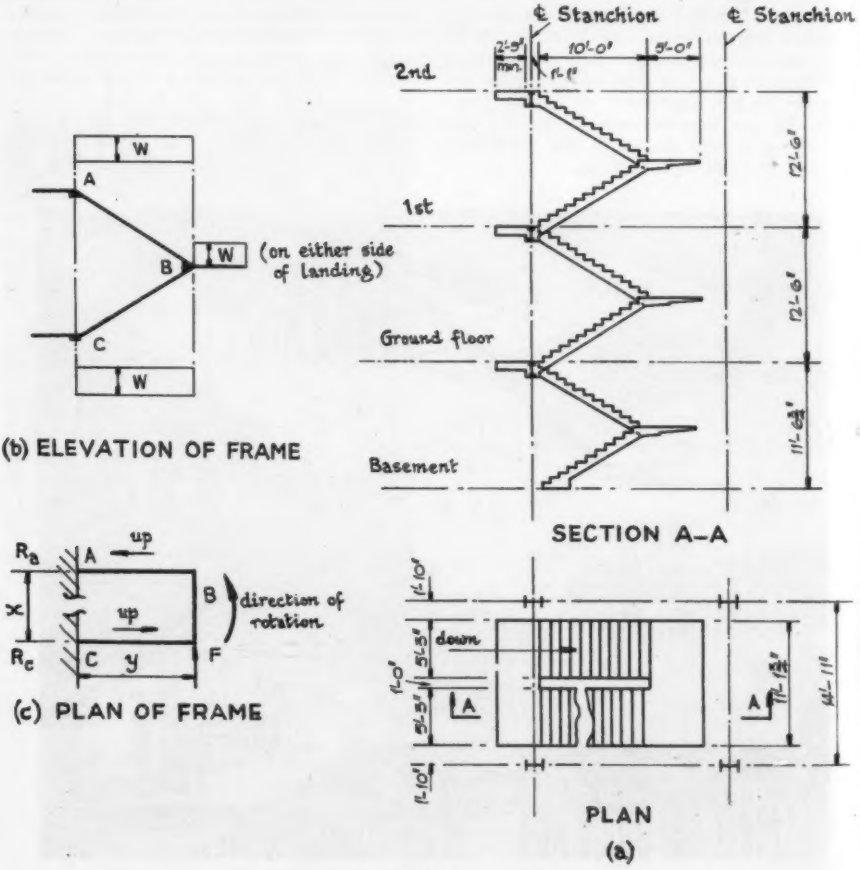


Fig. 2.

tedious, as great care was required in the selection of the combination of moments and forces for each case. The cantilevered half-landing was the only member not subjected to combined bending, torsion, shearing, and direct load. The results from this simplified method were in reasonable agreement with those obtained by the more complicated elastic theory.

Due to the high compressive strength of the concrete and the use of high-tensile steel, it is possible for the sizes of the concrete members to be small. The cantilevered landing is 5 in. thick at the root and 3 in. at the tip; the torsion beam at B between the stair flights is 21 in. wide with an average depth of 8½ in.; the stairs have a minimum thickness of 7¼ in.

As the stairs might be crowded in case of fire, particular attention was given to the fixing of the balusters for the handrails. The balusters fixed in the steps are secured in deep pockets reinforced with steel ties. The balusters for the half-landing pass through the slab, in steel tubes left very slightly proud of the concrete and anchored to it by ties, and are secured by nuts underneath and rest on flanges at the top of the slab. The force due to the tightening of the nut is resisted solely by the steel tube. The compressive strength of concrete cubes, tested after 28 days, exceeded 6000 lb. per square inch.

No appreciable vibration or deflection of the stairs has been apparent in use. A view of the completed stair is shown in *Fig.* 1. The architects were Messrs. C. J. Epril, F.R.I.B.A., & Associates, the contractors Messrs. Bovis, Ltd., and the consulting engineers Messrs. W. Hughes & Partners.

Drilling Concrete and the Building Regulations.

THE Building Regulations state that where a person is employed in the "cutting, dressing or carving . . . concrete" with mechanically-driven tools, goggles or screens must be provided to protect his eyes. In a recent case the defendant firm was prosecuted because of an injury to one of their employees who was drilling holes, with an electrically-driven hammer fitted with a bit which had four cutting edges, in a reinforced concrete ceiling. It was necessary for him to hold the hammer with the point of the bit upwards to make the hole, and while he was operating the tool a piece of steel and concrete fell from the ceiling and struck him in the eye and injured him. The defendant firm had not provided him with goggles.

The matter first came before the justices and the defendants argued that the work on which the injured man had been engaged was not the "cutting" of concrete within the meaning of the Regulations; they claimed that the word meant in that connection an alteration in the linear dimensions of the concrete in the same way as the other processes mentioned, namely, dressing and carving.

The justices came to the conclusion that the defendants were right and dismissed the summons, whereupon the prosecutor appealed to the High Court. The point of the matter, as that court remarked, was whether drilling a hole by a mechanical tool was "cutting". In this case, the court went on, the holes were being drilled by a tool with four sharp cutting edges that revolved at great speed. In the court's view, that was "cutting". To say that one cut a hole was inaccurate—the hole was made by cutting, but the hole was a void. The drilling of holes in concrete involved cutting, because the concrete was cut out in order to put something else in. There was no ambiguity in the word "cutting", and men's eyes should not be put in peril by attaching a narrow and technical meaning to it. The court accordingly declared that the justices' decision was incorrect and that there had been a breach of the regulations.

This decision is equally applicable to the drilling of stone and similar materials, and any doubt whether drilling operations of this sort are within the scope of the regulations [Regulation 84, Schedule 2, para. (2) of the Building (Safety, Health, and Welfare) Regulations 1948] has now been removed.

Book Reviews.

"Foundations Design and Practice." By Elwyn E. Seelye. (London: Chapman & Hall, Ltd. Price 128*s*.)

THIS book can be confidently recommended for use in the office of any civil engineer. It contains a very large number of diagrams and charts, and a fairly comprehensive series of design tables which may save much time. Illustrated alternative solutions are given to many problems, with notes on the factors to be considered and the dangers to be avoided. Examples are given of many types of failure, with notes on how to correct them. In addition to structural design, soil investigations, frost, seepage, earthquakes, underpinning, and construction methods generally are included. Almost every type of foundation structure is dealt with, including foundations for columns and walls, bridge piers and abutments, retaining walls, quays, dams, reservoirs, embankments, roads, and piling.

The illustrations and text are presented in a brief but clear manner. The work is not apparently intended for use as a textbook, but admirably shows how to deal with problems that might not otherwise come to an engineer's notice until after many years of office and site experience. As is usual with American books, the British reader will need to bear in mind differences of stresses, sieve numbers, legislation, and so on.—D. H. L. and C. E. R.

"Engineering Structural Failures." By Rolt Hammond. (London: Odhams Press, Ltd. 1956. Price 25*s*.)

SOMETIMES more can be learnt from a study of failures than from a study of successes, provided that the failures are not due to carelessness but to a misunderstanding of the effects of natural forces or to lack of knowledge of the strength of materials. Unless they are spectacular, failures do not usually receive much publicity. This book is consequently of value in that it contains summaries of many reports that are otherwise difficult to obtain. A large part of the book deals with earthworks, foundations, tunnels, and maritime structures, as may be expected, but there are also examples of failures of buildings and bridges. Amongst the latter is the Quebec bridge, on which there was a serious accident during the construction of the bridge replacing the original structure which collapsed through faulty design. Some methods of design to resist earthquake are outlined in a chapter which includes notes on the effects of vibration due to heavy machinery. Although there are several papers dealing with vibrational problems in structures, they are not well known to most engineers and there would appear to be scope for a book entirely devoted to this subject.

The reader will probably think of other examples that might well have been included. It would, for example, have been worth mentioning the collapse of the timber falsework of the Sandö bridge, reported to have been caused by an unexpected reduction in the strength of timber due to its dampness. Nevertheless, the examples which it contains make the book of interest to civil engineers.

"Building Elements." By R. L. Davies and D. J. Petty, M.B.E. (London: Architectural Press, Ltd. Price 37*s*. 6*d*.)

THIS is not a book on the detailed design and construction of the parts of a building. It is a discussion on the properties and uses of some building materials and of their use, and makes recommendations on the selection of materials and of the design of parts of a building for various requirements. The chapters deal with weatherproofing, insulation, fire protection, walls, roofs, floors, stairs, flues and fireplaces, windows, and doors, and there is a discussion, with illustrations, on modern architectural design.

"Plastic Methods of Structural Analysis." By B. G. Neal. (London: Chapman & Hall, Ltd. Price 45*s*.)

THIS is a treatise on the design of steel frames by the method developed in this country by Professor J. F. Baker at Cambridge University, and variously known as the plastic, ultimate load, and load-factor method. The analysis of steel structures by this method is fully dealt with, and is related to British Standard Code No. 449 of 1948.

Bow-string Bridges with Welded Tubular Reinforcement.

By PROFESSOR CH. SZÉCHY.

BUDAPEST.

FOR the rapid reconstruction of bridges in Hungary destroyed during the war structures were required that could be built quickly, using the materials that were available and that would need little timber. These conditions were largely fulfilled by composite constructions composed of structural steel plate girders and an overlying reinforced concrete slab and, for longer spans, by reinforced concrete bow-string girders with rigid reinforcement, both of which required little timber for staging or other purposes. In the case of the composite structures much of the steel was recovered from the old bridges. Also, steel tubes adaptable for rigid steel frames were available as surplus war material.

The first design using rigid tubular reinforcement was for the reconstruction of Margarete bridge * over the Danube at Budapest, comprising six steel arches with spans of 240 ft. to 290 ft. The rigid framework was already started when it was found that the bases of the piers were damaged and could not bear with safety the thrust of the heavier reinforced concrete arches. For this reason the bridge was reconstructed † with structural steel, and the steel tubes and the partly-fabricated rigid framework were used for other bridges. About six bridges with spans from 118 ft. to 201 ft. were constructed of this material. The most interesting features of these bridges are the following.

The arches were similar in shape to the funicular-polygon plotted for the total dead weight and half the live load. The rigid reinforcement comprised straight tubes arranged to form a polygonal Pratt truss, the deviation of the theoretical curve from the axes of the members being symmetrical. The chords were composed of four single tubes, or four double tubes for longer spans, and the connecting web-members were steel tubes of smaller sections (*Fig.* 1). All the connections were welded; the butt-welds along the intricate interpenetration curves of the circular sections required special care and supervision. The rigid frame was designed to carry the dead weight of the whole structure, including the weight of the concrete casing and reinforced concrete and of the deck, and in addition a pre-tensioning load equal to a part of the moving load. This was done because the steel in the framework is used most efficiently when the sections are stressed to the maximum as steel sections only, since when embedded in concrete as compressive members the stress in the steel cannot be greater than the permitted stress for concrete multiplied by the modular ratio, which is far less than the working strength of steel in compression. Also the stresses produced by the deformation of the steel framework may cause cracks in the reinforced concrete structure if this is not allowed for in the design; for example, the elongation of the tie-rod of a two-hinged arch reduces the horizontal thrust and causes an increase of the bending moments. This increase may be expressed as $\Delta M = \frac{3}{64} p . l^2 \nu$, and the ratio $\frac{\Delta M}{M_{max.}} = \frac{21}{8} \nu$, in which $M_{max.}$ is the maximum bending

* "Structural Engineer", 1937, page 110. † "L'Ossature Métallique", 1948, page 360.

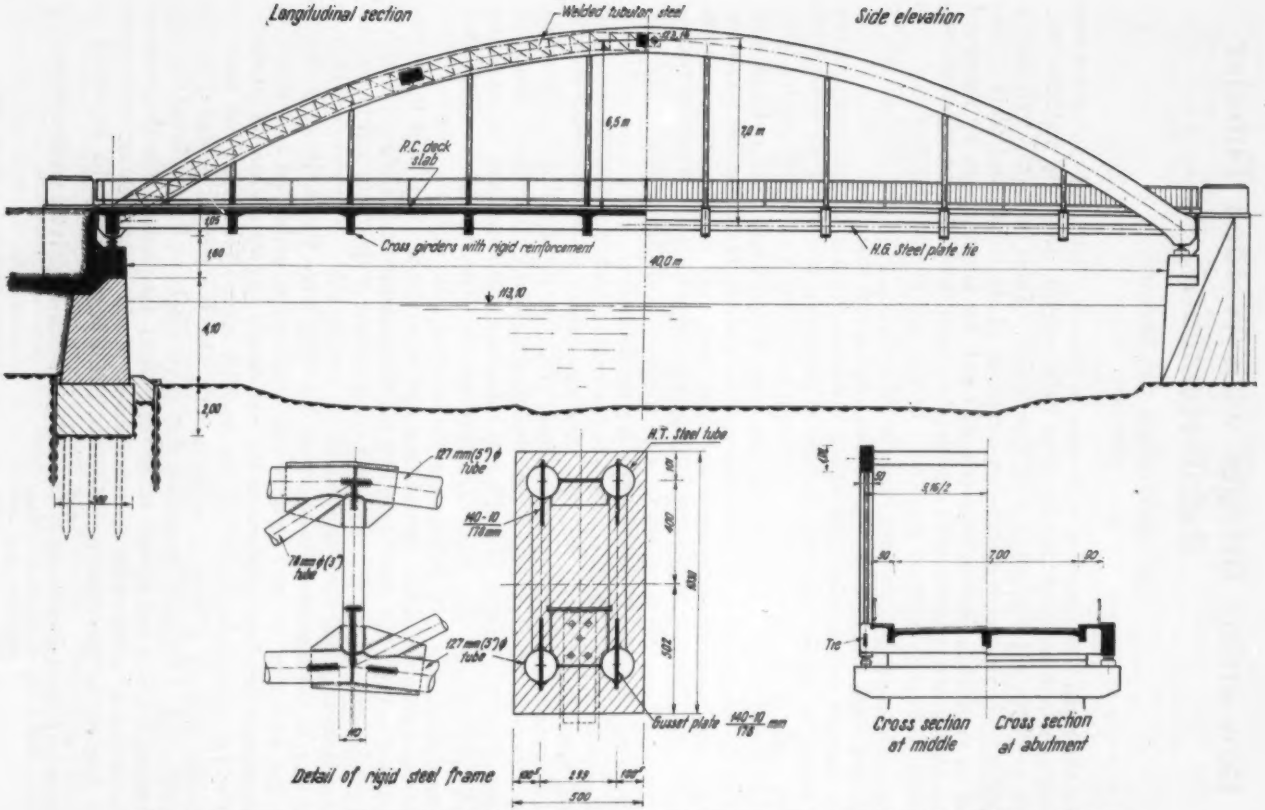


Fig. 1.—General arrangement of Rigid Tubular Reinforcement and Detail of Steel Plate Girders.

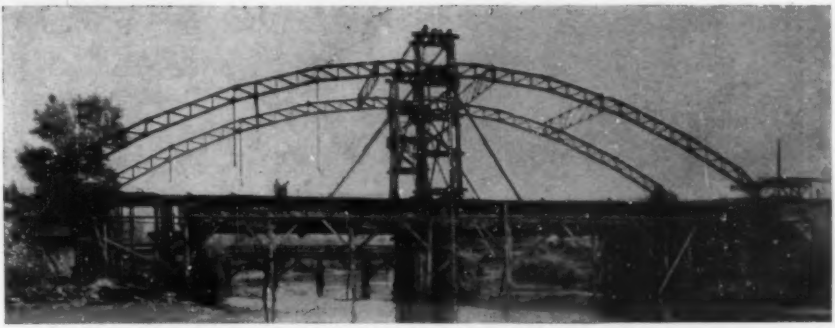


Fig. 2.—Light Staging used for Assembling and Welding Tubular Reinforcement.

moment at about one-quarter of the span of the arch, p is the intensity of the uniformly-distributed load, l is the span of the arch, and $v = \frac{15}{8} \cdot \frac{1}{f^2}\left(\frac{J_a}{F_a} + \frac{J_a}{F_t} \cdot \frac{E_b}{E_s}\right)$, in which f is the rise of the arch, J_a is the moment of inertia, F_a is the cross-sectional area of the arch, F_t is the cross-sectional area of the tie, and E_b and E_s are the values of Young's modulus for the concrete in the arch and the steel tie-rod respectively. The effect of this elongation may be neglected if the arch has three hinges and is statically determinate; therefore a temporary hinge was formed in the tubular steel frame at the crown. The effects of uneven loading and of unequal settlements were also counteracted in this way.

The elongation of the hangers is also a source of trouble, as the stretching of the steel may cause cracks in the encasing concrete: this can be reduced if the greater part of the load is applied before the concrete is placed, since the additional elongation which occurs when the rest of the load is applied is so small that no harmful cracks will be produced.

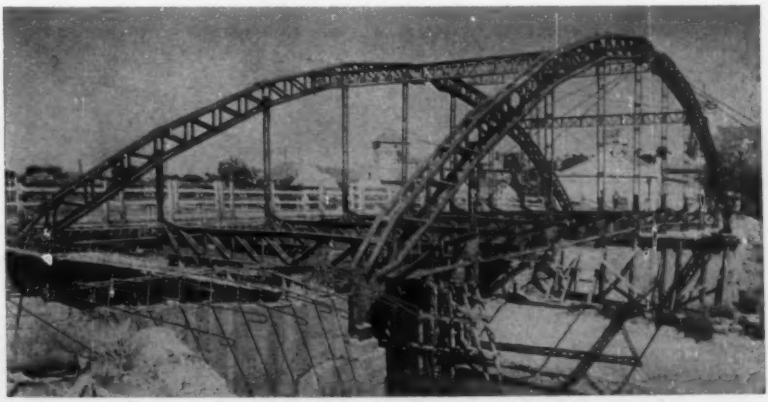


Fig. 3.—Tubular Reinforcement completed: note Cable acting as a Tie near the Springing.

Fig. 1.—General arrangement of Rigid Tubular Reinforcement and Detail of Steel Plate Girders.

Fig. 4.—Suspended Shuttering : note Steel-plate Tie-rod and Hangers.

These deformations were taken into account in the erection of the steel frame by giving the arch a suitable camber. The positions of the end bearings were also decided after considering the expected horizontal outward movement of the arch. When high-tensile steel cables were used instead of the mild steel tie-rods these deformations became considerable. In a case for which $l = 120$ ft. and $f = 20$ ft., the vertical movement of the temporary hinge at the crown increased from $1\frac{3}{4}$ in. to 9 in. and the elongation of the tie from $1\frac{1}{4}$ in. to 6 in. This fivefold increase was due to the reduction in the cross-sectional area of the tie resulting from an increase in the permissible stresses from 26,500 lb. to 114,000 lb. per square inch, and to the decrease of Young's modulus from 30,000 tons to 23,000 tons per square inch. These greater deformations also affect the value of the additional bending moments. In the first case ΔM did not exceed 2 per cent. of $M_{max.}$, but in the second case it was 26 per cent.

The economy of this type of construction depends on the work being carried out in the following order.

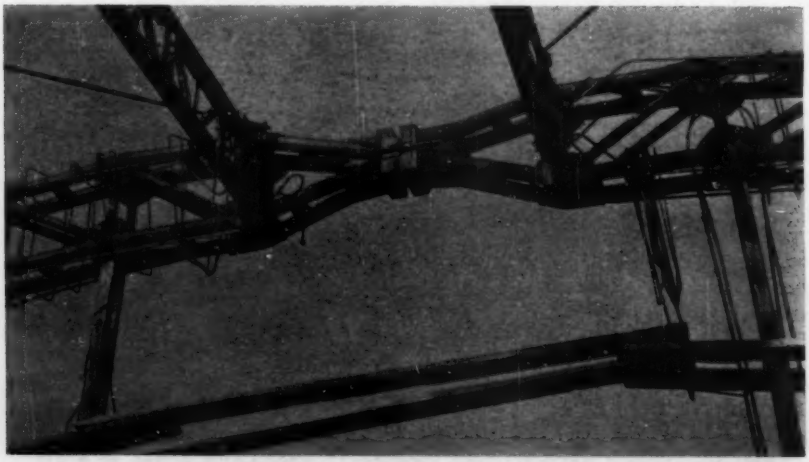


Fig. 5.—Temporary Hinge at Crown, Lateral Braces, and Hangers.

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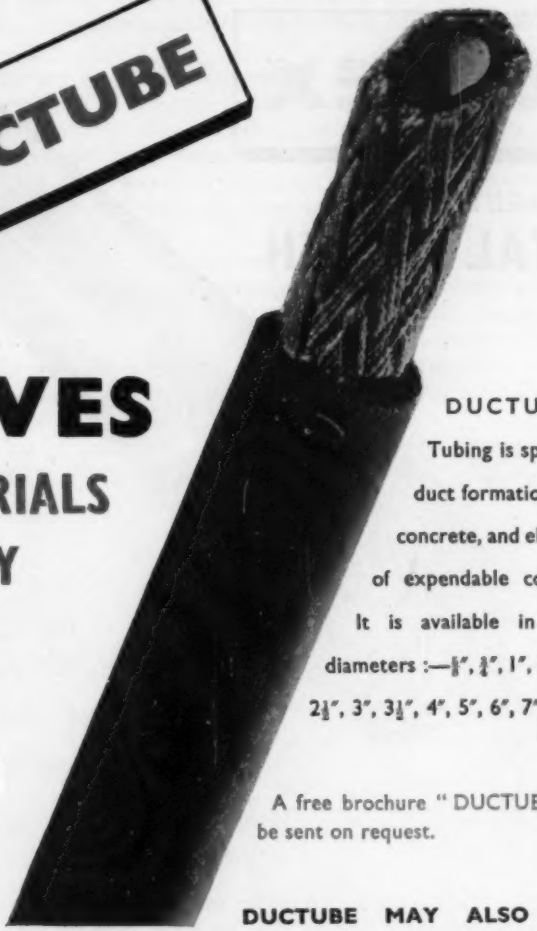

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1. The rigid truss of tubular steel is fabricated at the site in a horizontal position in halves. Provision is made for a temporary hinge at the crown between these halves.

2. The halves are erected in their correct positions, using a light staging (*Fig.* 2), and lowered on to the end bearings. The amount of camber and the position of the end bearings must be exactly as calculated, taking into account the free vertical and horizontal deformation and displacement of the structure.

3. The shuttering for the concrete arches and the supplementary normal reinforcement are fixed to the self-supporting rigid reinforcement and the concrete for the arches is placed (*Fig.* 4).

4. The concrete for the cross girders is placed at the same time and the reinforced concrete deck is then placed before the concrete in the arches has set. To permit the whole structure to move horizontally as the tie-rods stretch, a strip of deck-slab across the centre of the span is left unconcreted to allow free movement of the reinforcement bars which overlap at this point.

5. When the concrete of the deck is sufficiently strong, a load equal to half

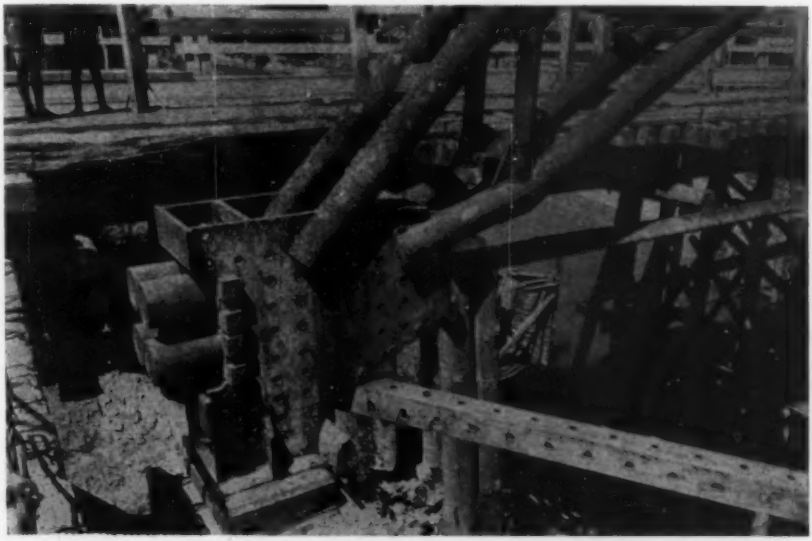

Fig. 6.—End Bearing, showing Welded Joint of Tubular Reinforcement and Anchorage of Tie.

the live load is placed on it and the strip at the centre of the deck, the hangers, and the temporary hinge at the crown are concreted. Experience has shown that when cables are used as ties the full live load should be applied before completing the placing of the concrete. It is also an advantage to leave gaps in the deck slab above the end-bearings, since the elongation of the ties causes angular distortion of the deck at these points.

Comparative Costs.

The economy of this method of construction is due to a reduction in the amount of timber and of staging required, and also in the construction time. The quantity of steel used is considerably greater than if bars were used, but the amount of concrete is slightly less.

A comparison can be made with a reinforced concrete bow-string bridge of 201 ft. span constructed just before the war, demolished during the war, and rebuilt using rigid tubular reinforcement. The quantities of materials used in the old bridge are given first and those used in the reconstruction are in brackets —Steel, 154,000 lb. (267,000 lb.); concrete, 471 cu. yd. (421 cu. yd.); timber, 355 cu. yd. (110 cu. yd.); construction time, 250 days (200 days). Fifteen per cent. of the steel in the new bridge was in the form of tubes. In this case, the timber required was reduced by 70 per cent. and the concrete by 11 per cent., but 75 per cent. more steel was used. The use of welded rigid tubular reinforcement will not therefore always be preferable, but it has decided advantages where the provision of scaffolding is difficult. For bridges with long spans it may compete successfully with prefabricated reinforced concrete construction.

The ties of this bridge were made of riveted mild steel plates with a total weight of 32,400 lb. (10·1 lb. per square foot of the area of the deck). The use of high-tensile steel cables would reduce this weight to 30 per cent. or about 10,000 lb. (3·4 lb. per square foot of the area of the deck). About 25 to 35 per cent. of the steel is required for the rigid tubular reinforcement and 12 to 18 per cent. for the ties.

Figs. 2 to 6 show details of the construction of these bridges including the method of erection and details of the anchorages for the ties, the temporary hinges, the end bearings, and the welded tubular trusses.

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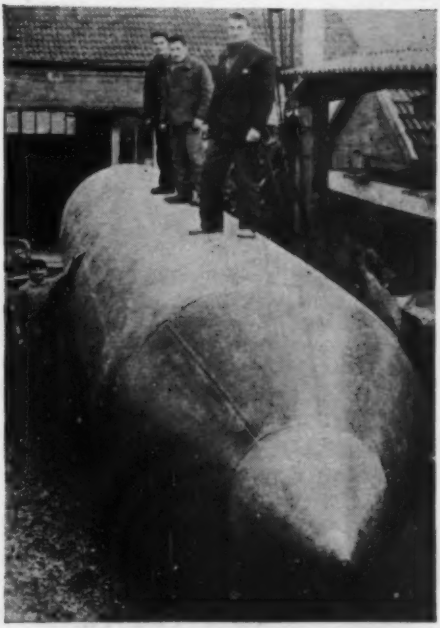

The photograph shows 87″ diameter CONTIDUCT, which was purchased for a syphon contract in Swaziland. It will be noted that the weight of the men produces no impression whatever, even though the amount of air required to inflate the tubing is only 2 lb. p.s.i. (approx.).

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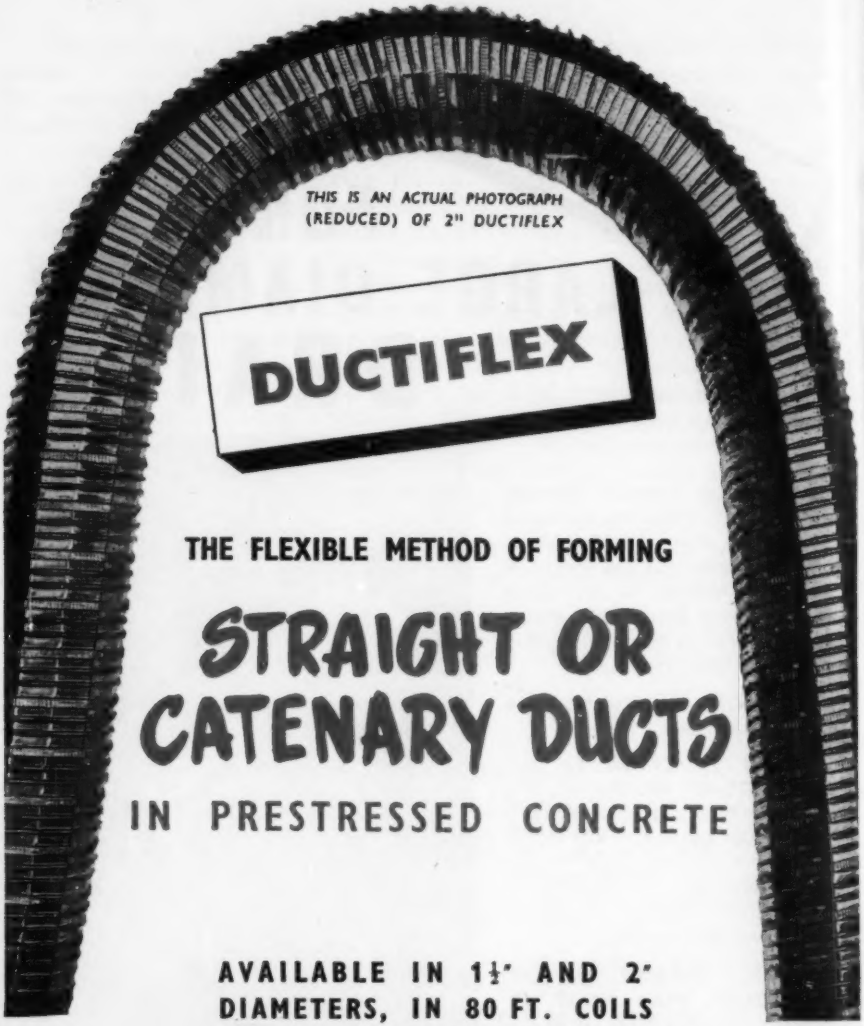

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Concreting in Cold Weather.

ENGINEERS from many countries where concreting has to be carried on at low temperatures attended a symposium on Winter Concreting held in Copenhagen last year under the auspices of the Danish National Institute of Building Research, and presented the results of their experience in overcoming this problem. These papers have now been published in a volume * of 1574 pages, which includes the contributions to the discussions which followed the papers. The volume is printed in the English language, and also includes an extensive summary of the principal conclusions reached printed in English, French, and German. Among the papers presented and contributions to the discussions published in this volume are :

WEATHER IN RELATION TO WINTER CONCRETING : Temperature-variations in Sweden in relation to tests of resistance to freezing, by Hans A. Vinberg. Calculation of the influence of the weather on concrete, by Jørn Jessing. Winter concreting in Poland, by M. Rzędowski. Contributions by J. G. Buitink, F. N. Sparkes, and E. G. Svenson.

EXPERIMENTS ON THE DETERMINATION OF THE RESISTANCE OF CONCRETE TO EARLY FREEZING : General Report, by Inge Lyse. Resistance to early frost action, by Göran Möller. Cold-weather concreting with high-early-strength cement, by Ernst Gruenwald. Effect of freezing on strength and expansion, by Arvo Nykänen. Tests of concrete at a temperature of 20 deg. F. (— 6·7 deg. C.), and Effect of entrained air and calcium chloride, by Lewis H. Tuthill. Freezing and thawing tests on green concrete, by Poul Nerenst and Niels Munk Plum. Influence of alcohol on concrete in cold weather, by Erik V. Meyer. Contributions by G. Möller, P. Lhopitallier, G. Wästlund, T. C. Powers, K. Schaden, A. Klein, and I. Lyse.

EFFECT OF TEMPERATURE ON THE HARDENING OF CONCRETE : General Report, by the Swedish Cement and Concrete Institute. Curing by electric heating, by K. Horimatsu. Computation of times for removing formwork, by Hans A. Vinberg. Effect of temperature on heat of hydration, by E. Rastrup. Hardening at different temperatures, especially below freezing point, by Arvo Nykänen. Effect of initial curing on strength, by T. Takahashi and M. Hayashi. Effect of low-temperature curing on compressive strength, by J. D. McIntosh. Hardening at different temperatures, by C. J. Bernhardt. Heat of hydration of cement, by U. Danielsson. Winter concreting in the Soviet Union, by S. A. Mironov. The use of the " Maturity "-function in assessing the sensibility of cements to low temperatures, by W. Brand. Hydration of cement as a function of temperature, by E. Rastrup. Contributions by E. Rastrup, P. Håkansson, W. Czernin, S. G. Bergström, T. N. W. Akroyd, M. Rzędowski, E. Lewicki, A. Klein, H. Granholm, and G. Wästlund.

RESISTANCE TO FROST AT EARLY AGES : General Report, by T. C. Powers. Influence of the age of concrete before exposure to freezing on strengths beyond 21 days, by J. Blondel. Computation of resistance to freezing at early ages, by P. Nerenst. Effect of air-entraining in cold weather, by V. V. Stolnikov. Evidences of disintegration of concrete affected by freezing and thawing, by Ervin Poulsen and G. M. Idorn. Contributions by T. C. Powers and W. Czernin.

ENSURING HIGH-QUALITY CONCRETE IN WINTER : General Report, by A. Voellmy. Television-tower at Stuttgart—a steel-concrete structure built in cold weather, by E. Bachus. Influence of heated mixing water on properties of concrete and heat of hydration of cement, by Hideo Yokomichi. Electrical curing in cold weather, by Yasuo Ichiki. Setting-temperature at air-temperature below freezing, by Chr. F. Grøner. Temperatures maintained by insulation, by Lewis H. Tuthill. Classification of durability according to resistance to frost, by S. V. Shestoperov. Heat treatment by steam at atmospheric pressure, by K. Thiel. The use of chloride salts, by S. A. Mironov and B. A. Krylov. Contributions by I. Grzymek, M. Kohn, P. Haller, F. Scheidegger, Inge Lyse,

* " RILEM Symposium on Winter Concreting." Obtainable from Concrete Publications, Ltd. Price 65s. (by post 67s.); 12.50 dollars in North America.

K. Böhmer, W. Grün, A. Lazard, A. Voellmy, J. Jambor, H. Rüsch, J. D. McIntosh, F. Hess, and A. Staub.

CONSTRUCTION IN COLD WEATHER: Winter concreting in Finland, by Beato Kelopuu. Winter concreting in Holland, by J. G. Buitink and J. M. L. Trouw. Construction of houses and multiple-story buildings in cold weather, by Arvo Nykänen. Use of ready-mixed concrete in winter, by Folmer Jørgensen. Winter concreting in China, by L. S. Wu. The hardening of concrete in frost, by V. N. Sizov. Electrical heating of massive structures such as dams, by K. V. Alexeyev. Contributions by H. Rühle, J. J. Bouvy, K. Schaden, J. Talbierski, A. Klein, and F. W. Katlein.

A Prestressed Footbridge.

A PRESTRESSED concrete footbridge more than 600 ft. long has been built to the east of Scunthorpe station by British Railways, Eastern Region. There are five spans with a total length of 373 ft., and approach ramps about 125 ft. long. The overall width is 7 ft. 4 in., and the depth 6 ft. Sulphur-resisting cement and granite aggregates were used as a protection against industrial fumes.

The prestressed beams are 6 ft. deep and are prestressed with pre-tensioned wires; they vary between 67 ft. and 86 ft. in length, and are supported on reinforced concrete precast trestles. The beams and trestles of the approaches are of reinforced concrete. The trestles over the tracks are 19 ft. high and 15 ft. wide at the base.

Between the longitudinal beams are transverse beams, 4 ft. 6 in. long, 1 ft. 9 in. wide, and 11 in. deep, at intervals of about 8 ft. The transverse beams are prestressed with post-tensioned steel bars which project from the ends and pass through holes in the main beams; these bars are anchored by nuts, which were tensioned by spanners to 125 ft.-lb. and grouted.

The deck comprises hollow slabs 4 ft. 7 in. long by 1 ft. 2 in. wide and 4 in. thick bedded in mortar. The trestles, weighing between 6 and 8 tons, were cast complete, as were the main beams which weigh between 30 and 35 tons each. The main beams were transported in pairs by rail and erected at week-ends. Each span was erected in twelve hours, with the use of two breakdown cranes to lift the beams from the wagons and place them in position. The lifting cradles incorporated a universal joint in the uprights.

The bridge was built under the general direction of Mr. A. K. Terris, B.Sc., M.I.C.E., Chief Civil Engineer, Eastern Region. The consulting engineers were Messrs. Rendel, Palmer & Tritton. The contractors were Dow-Mac (Products), Ltd., of Stamford, who made the precast parts.

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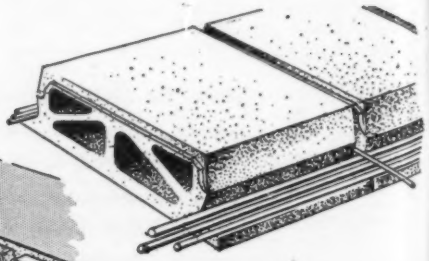

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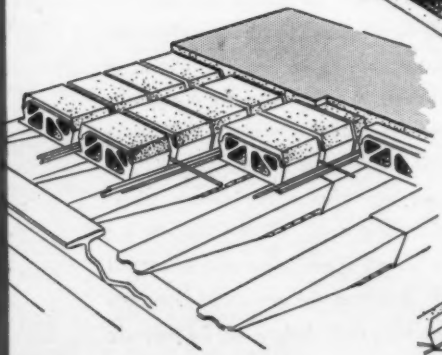
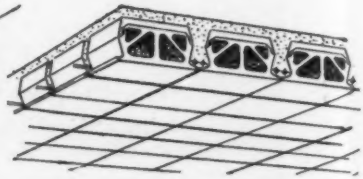


Showing Two-Way Reinforcement and Hollow Concrete Blocks laid on Trianco Telescopic Centers.



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Prestressed Footbridge at Twickenham.

A PRESTRESSED precast footbridge has been built over the river Thames at Eel Pie island, Twickenham, Middlesex, to replace a ferry. The bridge (*Fig.* 1) has a central span of 84 ft. with two spans of 34 ft. and 10 ft. 2 in. at one end and one span of 42 ft. at the other end. The unusual rise of the arch provides the clearance for navigation required over 60 ft. of waterway without long approach ramps. The piers are built on precast concrete piles, and consist of solid precast sections grouted together and chamfered at each end so as to reduce obstruction to the stream. The deck is formed of beams cast in short lengths at a factory, assembled on scaffolding, and prestressed by the Gifford-Udall-CCL system to form a continuous structure. The parapets are of wood.

The deck consists of two tee-beams (*Fig.* 3), made of precast units 8 ft. long, joined transversely by precast slabs of reinforced concrete 4 in. thick, and prestressed to form a continuous structure supported on concrete abutments cast in place and on the precast piers; they are fixed at one end and are on precast concrete rocker-bearings at the three other supports. The ends of the beams are held down by prestressing wires passing through the end anchorage blocks and cast into the abutment at the fixed end and passing through the precast rocker-bearing at the free end.

The piers are founded on two blunt-ended reinforced concrete piles 12 in. by 12 in. by 20 ft. long. The piers were precast in three parts, each weighing 3 tons. The piles and the pier units are

Fig. 1.

Fig. 2.

joined by mild steel bars grouted into vertical ducts of 6-in. diameter.

The deck beams are designed for a live load of 60 lb. per square foot. On a centering of scaffold tubes (*Fig.* 2) driven into the river-bed by hand, the precast units were assembled, with joints not less than 1 in. wide which were caulked with stiff 1 : 1 mortar. At the crown the width of the joints was increased to about 2 in. through the ducts with a hand winch. The anchor-blocks were cast in two parts and joined by transverse prestressing. Tensioning of the wires was carried out from each end. The loss of prestress due to friction was assumed to be 20 per cent.; the actual loss was 16 per cent.

After stressing, neat cement grout, with the addition of a chemical to produce

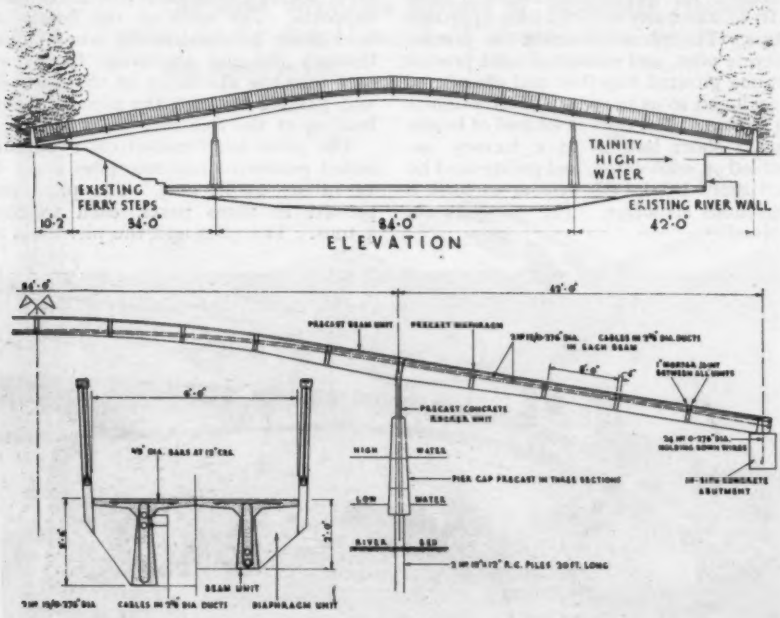

Fig. 3.

and it was found necessary to reinforce them with a ¼-in. mild steel bar and to leave a soffit shutter-board under the joint until it had hardened. The cable ducts were formed through the joints by short lengths of flexible tube connected to ¼-in. air-hose.

Each beam has two cables of twelve 0·276-in. wires, which are continuous and looped at one end; they were drawn expansion, was injected from each end to a vent-hole at mid-span. The total construction time was three months.

The consulting engineer was Mr. E. W. H. Gifford. The main contractors were Messrs. Reed & Mallik, Ltd., the contractors for the piling were Piling & Plant (Kent), Ltd., and the precast concrete units were made by the Liverpool Artificial Stone Co., Ltd.

the right cure for bacon is oak chippings . . .

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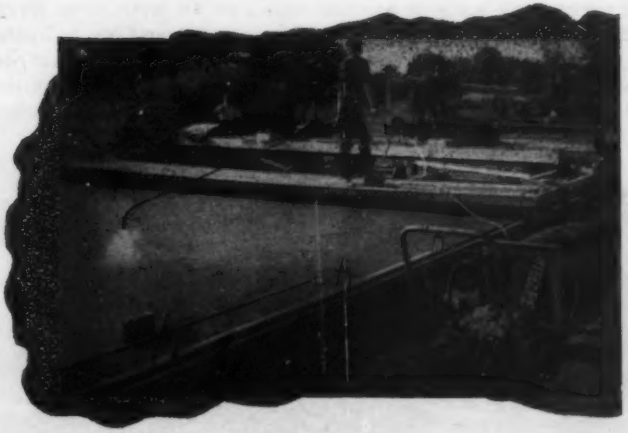

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New concrete road construction technique

The first Winget-Vibrocon B.U. 52 Concrete Spreading Machine was ordered by the Kent County Council (on the recommendation of the Road Research Laboratory and in conjunction with the Ministry of Transport and Civil Aviation) for use on the construction of the Ashford By-Pass. This picture, taken on a trial run at Potter's Corner, Hothfield, half a mile distant, shows the machine at work. Here, concrete is supplied by a battery of three Winget 10 HTH Hydraulically Operated Weigh Batching Tilting Drum Mixers feeding concrete to high loaders, which in turn charge a tipping lorry sited at the side of the spreader. The Winget-Vibrocon B.U. 52 Spreading Machine not only travels along the formwork spreading low-slump concrete with the smallest possible fall (to prevent pre-compaction and segregation), but gives accurate control of the depth of the spread carpet of concrete. The machine is powered by Diesel Electric Engine and has hydraulic control for varying the carpet thickness. Only one operator is required, and when it is necessary to remove the machine to another site, it can be telescoped to a size suitable for movement on public roads, and fitted with pneumatic tyred wheel assemblies. (Note Winget 'Vibrocon' Poker Vibrator in front of spreader.)

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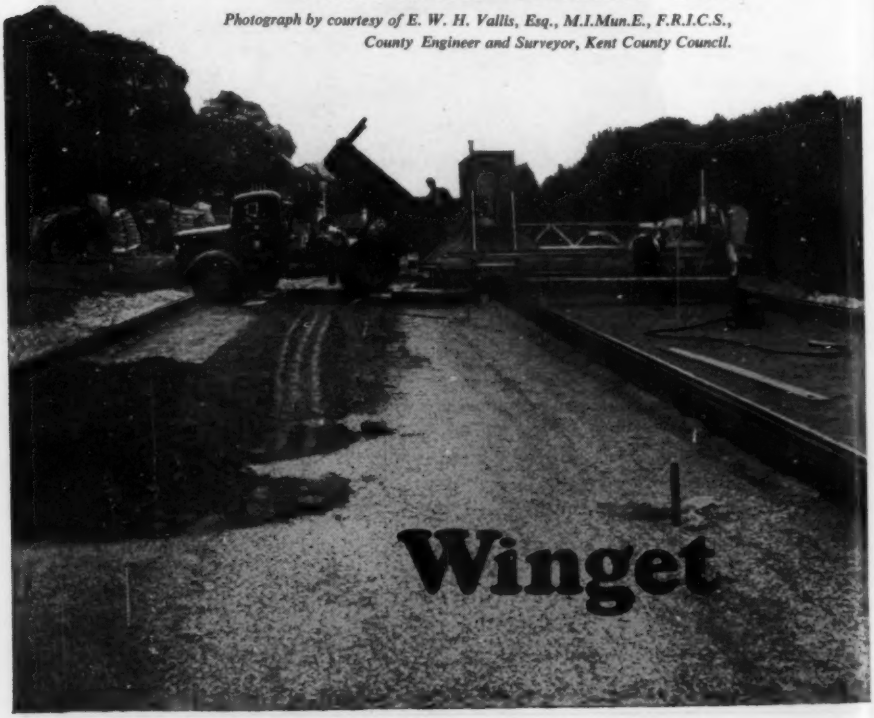
Photograph by courtesy of E. W. H. Vallis, Esq., M.I.Mun.E., F.R.I.C.S., County Engineer and Surveyor, Kent County Council.



A Precast Factory.

A FACTORY (*Fig.* 1) at Eastleigh, Hampshire, for Messrs. William R. Warner & Co., Ltd., was built and ready for occupancy in twelve months. The structure comprises a single-story warehouse, a two-story manufacturing building, and a single-story office and canteen building, all of which are built of precast members. The warehouse has an area of 63,000 sq. ft., the manufacturing building 13,200 sq. ft., and the office building 10,800 sq. ft.

The foundations of all the buildings are plain concrete blocks on subsoil capable of sustaining 2 tons per square foot. Pockets were formed in the foundations to accommodate the columns, which were grouted in position with a 1 : 3 cement-sand mortar.

WAREHOUSE.—The warehouse, which is in reinforced concrete, is 17 ft. 6 in. high and has a flat roof 320 ft. by 198 ft. The floor is 6 in. thick. The columns are at 32 ft. by 22 ft. centres. Erection of the frame was completed in less than eight weeks at a cost of 5*s.* 3*d.* per square foot of floor area, or 3·35*d.* per cubic foot. The main beams are 7 in. wide and 16 in. deep. The purlins are at 8 ft. centres and are of T section with thickened ends. A degree of continuity in the purlins was achieved after erection by laying deformed bars, 4 ft. long by 2¼ in. diameter, in grooves across the junctions; the grooves and the gaps between the purlins were filled with mortar.

On three sides the precast concrete gutters, and on one side a kerb, project beyond the frame to provide a fixing for metal-framed glazing at the top of the

Fig. 1.—General View.

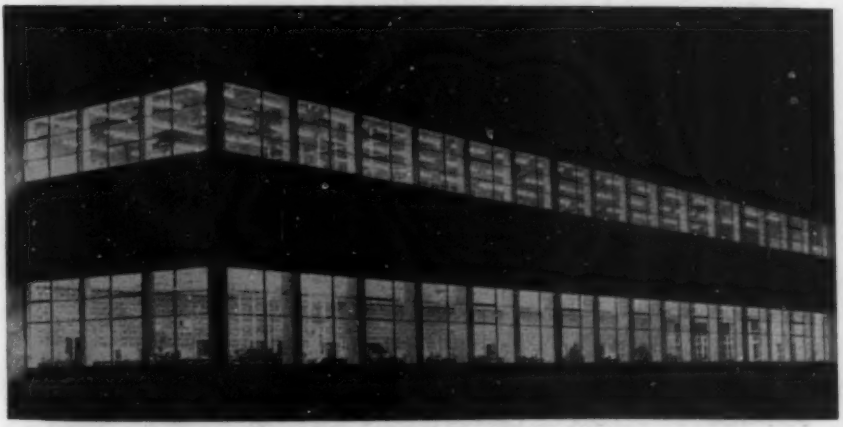


Fig. 2.—The Manufacturing Building.

walls. The remainder of the walls comprises lightweight reinforced concrete slabs measuring 2 ft. by 13 ft. 3 in. by 9 in. The aggregate exposed on the white facing concrete is ⅛-in. calcined flint.

MANUFACTURING BUILDING. — The manufacturing building is 198 ft. long by 68 ft. wide, and in both stories there is a line of columns dividing the floors into widths of 19 ft. and 48 ft. The lower story is 17 ft. 6 in. high and the upper story is 18 ft. 3 in. high. The columns are at 11-ft. centres, and are generally 1 ft. 8 in. by 1 ft. in cross section in the lower story and 1 ft. square in the upper story. The main beams in the first floor are 3 ft. deep by 10 in. wide and span 48 ft. Two beams are used over the columns and were bolted together after erection. The main beams in the roof are 2 ft. deep by 1 ft. wide. All the beams were prestressed on the "long line" system with pre-tensioned wires. Expansion joints are provided 66 ft. apart; the joints were formed by erecting two columns ½ in. apart, placing one beam on each column, and filling the continuous gap with jointing material. The walls are of precast blocks faced with brick; the interior wall between the columns is also of concrete blocks.

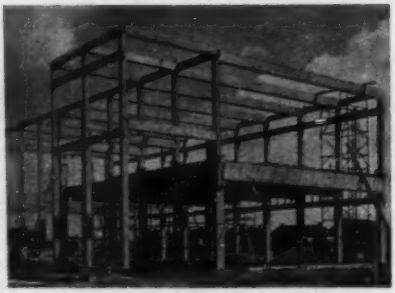


Fig. 4.—Frame for Manufacturing Building.

ADMINISTRATION BUILDING.—The administration building is 176 ft. long and partly 73 ft. and partly 52 ft. wide. In the office area are three lines of nine columns 15 ft. high and 10 in. square at 12 ft. 9 in. centres. The main beams are 1 ft. 3½ in. by 10 in. in cross section and span 20 ft. 4 in. and 23 ft. The side and edge-beams are 9 in. deep by 6½ in. wide. In the canteen portion there are three lines of six columns, 1 ft. by 1 ft. 3 in. in cross section, at 12 ft. 9 in. centres; the main beams are 1 ft. 9½ in. deep by 1 ft. wide and span 36 ft. 3 in. and 36 ft. 7½ in. The roof is of prestressed hollow beams

Fig. 3.—Columns and Beams for Warehouse.

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STAHLTON FLOORS AND ROOFS were introduced into this country by the Costain organization in 1952.

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with a 2-in. topping of vermiculite concrete and a layer of bituminous felt. The walls are of concrete blocks faced with brick.

The same method of construction is being used for an extension of the warehouse (154 ft. by 132 ft.) and of the manufacturing building, and a story is being added to the office building.

The architects are Messrs. Edward D. Mills & Partners and the consulting

Fig. 5.—Precast Wall Slabs.

engineer Mr. C. V. Blumfield, B.Sc.(Eng.), M.I.C.E. The general contractors are Messrs. Holland & Hannen and Cubitts, Ltd. The frame of the warehouse was designed, made, and erected by Messrs. R. E. Eagan, Ltd., the prestressed beams were made by the Concrete Development Co., Ltd., and the exposed aggregate wall slabs were made by Cheecol Processes, Ltd.

DATA FOR PRICING REINFORCED CONCRETE.

Materials.

(Delivered in London area.)

AGGREGATES (per cu. yd.).—Washed sand, 27*s.* 7*d.* Clean shingle: ¾ in., 23*s.* 11*d.*; ½ in., 26*s.* 6*d.* Pit ballast, 27*s.*

CEMENT (per ton, delivered at Charing Cross).—Portland cement, 6 tons and upwards, 104*s.* 6*d.* 1 ton to 6 tons, 116*s.* 6*d.* Paper bags and non-returnable jute sacks included.

Rapid-hardening Portland, 10*s.* 6*d.* above ordinary Portland.

Aquacrete and 417, 32*s.* 6*d.* above ordinary Portland; paper bags included.

Colorcrete (buff, red, and khaki), in 6-ton loads, 161*s.*; paper bags included.

Snowcrete, 265*s.*, including paper bags.

"Super-Cement," 32*s.* 6*d.* per ton above ordinary Portland cement; paper bags included.

High-alumina cement: per ton, 314*s.* 6*d.*; 2 tons, at 304*s.* 3*d.*; 4 tons, at 299*s.* 3*d.*; 6 tons, at 295*s.* 9*d.*

Snowcem paint, 71*s.* per cwt. including containers.

SHUTTERING.—1″ and 1¼″ deal sawn shuttering boards, £95–£100 per standard.

REINFORCEMENT.—Mild steel bars, B.S. 785 (per cwt.); ⅜ in. to 2½ in., 45*s.* 9*d.*; ⅛ in. to ½ in., 47*s.* 6*d.*; ⅝ in., 48*s.* 3*d.*; ¼ in., 49*s.* 9*d.*

Materials and Labour.

(Contracts up to £5000. Including 10 per cent. profit.)

PORTLAND CEMENT CONCRETE, 1 : 2 : 4.—Foundations, 2*s.* 10*d.* per cu. ft. Columns, 3*s.* 10*d.* per cu. ft. Beams, 3*s.* 7*d.* per cu. ft. Floor slabs, 4 in. thick, 10*s.* 2*d.* per sq. yd.; Do., 5 in., 12*s.* 2½*d.*; Do., 6 in., 14*s.* 8*d.*; Do., 7 in., 17*s.* 3*d.* Walls 6 in. thick, 16*s.* 3*d.* per sq. yd. Add for hoisting above ground level, 3*s.* 10½*d.* per cu. yd. Add for rapid-hardening Portland cement, 3*s.* 10½*d.* per cu. yd.

REINFORCEMENT.—Mild steel bars (B.S. 785), including cutting, bending, fixing, and wire (per cwt.)—½ in. to ¾ in., 90*s.* ⅜ in. to ½ in., 80*s.* 10*d.* ⅛ in. to 2½ in., 72*s.* 8*d.*

SHUTTERING AND SUPPORTS.—Walls, 224*s.* 6*d.* per square.

Floors (average 10 ft. high), 208*s.* 6*d.* per square. In small quantities, 2*s.* 9*d.* per sq. ft.

Columns, average 18 in. by 18 in. (per sq. ft.), 3*s.* 1½*d.*; in narrow widths, 4*s.* 0½*d.*

Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 3*s.* 2*d.*; in narrow widths, 3*s.* 8*d.*

Raking, cutting, and waste, 7*d.* per lin. ft.

Labour on splays, 3½*d.* per lin. ft.

Small fillets to form chamfers, 7*d.* per lin. ft.

Wages.

The rate of wages on which the above prices are based are: Carpenters and joiners, 4*s.* 6*d.* per hour (carpenters 2*d.* a day tool money); Labourers, 3*s.* 11½*d.*; Men on mixers and hoists, 4*s.* 2½*d.*; Bar-benders, 4*s.* 3*d.*

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Hydro-electric Works in Scotland.

THE North of Scotland Hydro-Electric Board is intending to use the waters of the rivers Farrar and Beauly for the production of 268,000,000 units of electricity annually. The catchment area comprises 350 square miles in the counties of Inverness and Ross and Cromarty, and there will be four power stations with a total capacity of 102,000 kW. At present prices the estimated cost is £14,250,000, and the works will take about five years to complete.

The available fall in the river Farrar will be developed at two power stations. The upper section will include a dam 110 ft. high, and there will be a subsidiary dam 50 ft. high. From Loch Monar the water will pass through a tunnel 5¾ miles long to a power station near Deanie.

The level of Loch Beannachran will be raised about 8 ft. by means of a small dam and a tunnel 3½ miles long. It is also proposed to pass the waters of the river Beauly through two power stations. This will require the construction of two dams, each about 50 ft. high with a power station of about 20,000 kW. capacity built into it.

Two Contracts Awarded.

CONTRACTS for the construction of tunnels and for a power station have been awarded in connection with the Scottish hydro-electric schemes. One contract valued at £2,500,000 is for the construction of over thirteen miles of tunnels in the Killin area of the Breadalbane scheme in Perthshire for the North of Scotland Hydro-electric Board. This contract includes the construction of pipeline aqueducts eight miles in length, some miles of roads, and intake weirs and shafts. The consulting engineers are Messrs. James Williamson & Partners.

The second contract, valued at £1,250,000, is for a generating station at Kincardine, Fife, for the South of Scotland Electricity Board, and includes the superstructure of the main building, the turbine house, boiler house, administration, workshop, and other buildings. The consulting engineers are Messrs. Kennedy & Donkin.

Both contracts have been awarded to the Mitchell Construction Company.

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Department of Civil Engineering

Bursaries in Concrete Technology

NOTICE IS HEREBY GIVEN that the election to Bursaries in Concrete Technology, tenable as from October, 1957, will take place in June, 1957.

Candidates must hold a degree in Engineering at the time of taking up the award, and must also have a good knowledge of the theory of structures.

Bursaries are of the value of £460, out of which the College Tuition Fee of £60 has to be paid; the amount may be increased to a maximum of £760, depending on the qualifications and length and nature of industrial experience. The Bursary is for the normal College session extending from October until the following June.

The course will be postgraduate, and Bursars who successfully complete the course will be eligible for the award of the Diploma of the Imperial College (D.I.C.).

Applications must be received on or before 1st June, 1957, by the DEPUTY REGISTRAR, CITY AND GUILDS COLLEGE, Exhibition Road, London, S.W.7, who will, on written request, send full information and application forms.

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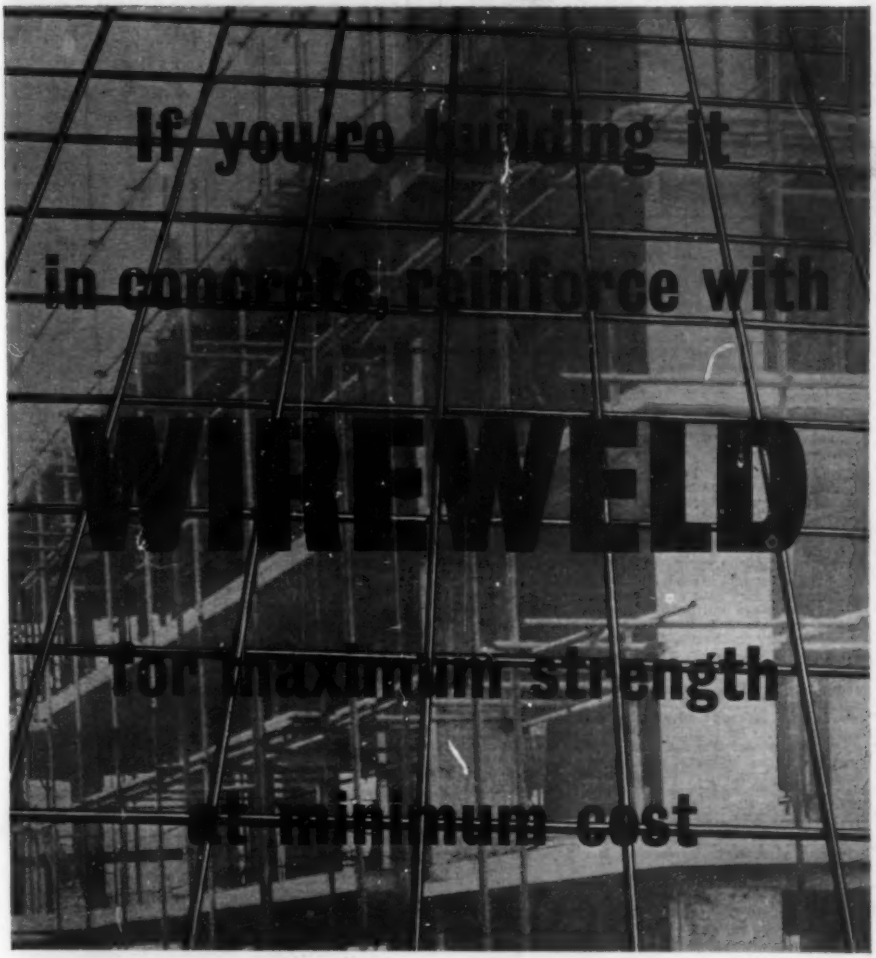

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SITUATION VACANT. Reinforced concrete designer-draughtsman for work in Sunbury or London office. Five-days' week. Salary £800–£1000 according to experience. Apply, stating age and details of experience, to J. H. Coombs & Partners, Consulting Engineers, Thames Corner, Sunbury-on-Thames, Middx.

SITUATIONS VACANT. R. L. Bourqui & Partners, Thermix House, Kingston-on-Thames, require detailers immediately for reinforced concrete work. Only first-class draughtsmen with good printing and linework considered. Knowledge of design an advantage, but ability to detail from calculations essential. Should be able to construct general arrangement drawings from sketches of industrial structures, framed buildings, etc. Above-average salaries offered to competent and conscientious men. Full holidays this year. Telephone for appointment to Elmbridge 0439 (office hours) or Elmbridge 2634 (evenings).

SITUATIONS VACANT. Consulting engineers require reinforced concrete designers and detailers for varied and interesting structures. Opportunity for experience in steelwork. Vacancies in offices in Central London and in Birmingham. Five-days' week, good conditions. Applications in strict confidence, stating full details including salary required, to Alan Marshall & Partners, 73 High Holborn, London, W.C.1.

SITUATIONS VACANT. Assistant structural engineers required with experience of reinforced concrete design. Experience of precast techniques advantageous. Salary according to experience and qualifications. Apply in writing to Technical Manager, The Scottish Construction Co. Ltd., Sighthill Industrial Estate, Edinburgh.

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SITUATIONS VACANT. Acrow require draughtsmen and designer-draughtsmen, age around 25 years, for their steel formwork division. Vacancies occur due to existing staff being promoted to important positions in our Canadian company. Highest salaries paid, and excellent prospects of advancement afforded to competent men capable of working on own initiative. Apply, giving full details of experience, and commencing salary required, to Mr. R. J. TOOLEY, ACROW (ENGINEERS) LTD., South Wharf, Paddington, London, W.2.

SITUATION VACANT. Reinforced concrete detailing draughtsman, with knowledge of design an advantage, required by consulting engineer engaged in all types of structures. Salary £500–£750 according to experience. Write, stating full particulars, to P. R. ROSE, A.M.I.Struct.E., 7 Connaught Place, London, W.2.

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SITUATION VACANT. Civil engineering draughtsman required for work in London office of large civil engineering contractors. Work of varied nature, with scope for initiative and advancement. Commencing salary £600–£800 depending on experience. Five-days' week and luncheon vouchers. Pension scheme operates. Box 4228, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Reinforced concrete designer-detailer required by contractor to assist in building up a new department. Qualified men experienced in reinforced concrete detailing and also able to supervise draughtsmen offered better-than-average salary and permanent pensionable post. Apply, in writing, to CHARLES R. PRICE, Barnsley Road, Doncaster.

SITUATION VACANT. Concrete inspector or engineer required initially to supervise manufacture of high-quality precast prestressed bridge beams at factory and later to assist on site of major road and bridge scheme in London area. Salary £800 per annum. Write, stating age and details of experience, to HARRY BROMPTON, Consulting Engineer, 7 Hobart Place, Westminster, S.W.1.

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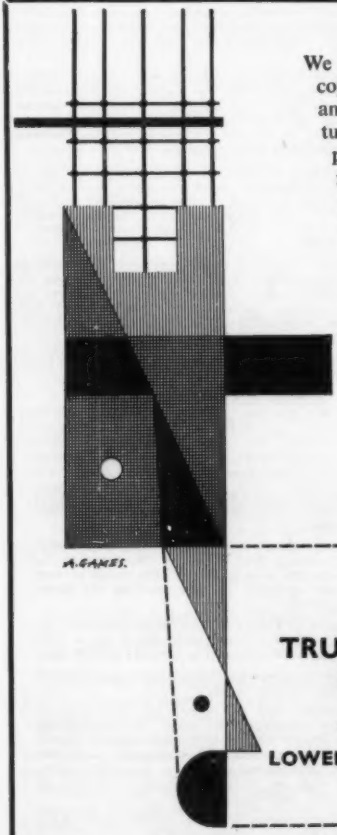

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